WASATCH REGIONAL LANDFILL

MUNICIPAL SOLID WASTE LANDFILL PERMIT MODIFICATION

HAND DELIVERED

DEC 2 7 2004
04.04384
UTAH DIVISION OF
SOLID & HAZARDOUS WASTE

DESIGN ENGINEERING REPORT

DECEMBER 2004

HANSEN
ALLEN
& LUCE INC

ALLIED WASTE SERVICES

HAND DELIVERED

JUN 27 2005
05.02184
UTAH DIVISION OF
SOLID & HAZARDOUS WASTE

June 27, 2005

Dennis R. Downs
Utah Department of Environmental Quality
Division of Solid and Hazardous Waste
288 North 1460 West
Salt Lake City, UT 84114

RE: Updated Closure and Post-Closure Tables

Dear Mr. Downs,

Please include these updated tables in the Municipal Solid Waste Landfill Permit Modification Closure, Post-Closure Care And Financial Assurance Plan that was previously submitted on June 22, 2005.

If you have any questions please contact me at 801-924-8485

Sincerely,

Les Lemmon

Lester Lemmon
Operations Manager
Wasatch Regional Landfill, Inc.

TABLE 1 Wasatch Regional FINAL YEAR 3 CLOSURE COST ESTIMATES SUMMARY				
SIZE OF CLOSURE AREA:	30.0 ACRES			
CLOSURE COSTS	MEASURE	UNIT COST	QUANTITY	TOTAL
Supply & Placement of Closure Cap				
General Contractor Mobilization/Demobilization (1)	Lump Sum	$ 25,000.00	1	$ 25,000.00
Liner Contractor Mobilization/Demobilization (1)	Lump Sum	$ 5,000.00	1	$ 5,000.00
GCL (1)	Acre	$ 17,563.00	30	$ 526,890.00
60 Mil HDPE Textured (1)	Acre	$ 20,440.00	30	$ 613,200.00
Freight and Material Taxes (1)	Included in $/Acre			
Install Gas Vents (100' grid spacing)	EA	$ 300.00	100	$ 30,000.00
Soil Cover (21") (1)	Acre	$ 13,419.00	30	$ 402,570.00
Grading of Waste/Surface Preparation (3)	Acre	$ 1,000.00	30	$ 30,000.00
Surveying (4)	Acre	$ 500.00	30	$ 15,000.00
Stone Mulch (1)	Acre	$ 940.00	30	$ 28,200.00
Subtotal				**$ 1,675,860.00**
Stormwater/Groundwater Controls				
Channel Excavations (3)	LF	$ 2.00	2500	$ 5,000.00
Riprap Channel Granular Filter (run-on control) (3)	CY	$ 10.00	1400	$ 14,000.00
Riprap Channel Riprap (run-on control) (3)	CY	$ 50.00	2100	$ 105,000.00
Downchute Pipe (3)	LF	$ 57.50	400	$ 23,000.00
Inlet Boxes (3)	EA	$ 2,500.00	2	$ 5,000.00
Install Remaining Groundwater Drain (3)	CY	$ 1.50	63355	$ 95,032.50
Install Drain Pipe Under Railroad (3)	Lump Sum	$ 85,000.00	1	$ 85,000.00
Subtotal				**$ 332,032.50**
Leachate Evaporation Pond (assume approximately 100' x 100' x 10' deep)				
Pond Excavation/Earthwork (1)	CY	$ 2.00	1850	$ 3,700.89
GCL (1)	Acre	$ 17,563.00	0.3	$ 5,268.90
60 Mil HDPE Textured, 3-layers (1)	Acre	$ 20,440.00	0.9	$ 18,396.00
Geonet, 2-Layers (1)	Acre	$ 10,481.00	0.6	$ 6,288.60
Freight and Material Taxes (1)	Included in $/Acre			
Leak Detection Pipes and sumps (1)	EA	$ 10,000.00	2	$ 20,000.00
Subtotal				**$ 53,654.39**
Other: (List)				
Engineering Site Evaluation (4)	LS	$ 10,000.00	1	$ 10,000.00
Design, Specification & CQA/CQC Manual (4)	LS	$ 50,000.00	1	$ 50,000.00
Project Mgmt. & QA/QC, Oversight (4)	Acre	$ 2,500.00	30	$ 75,000.00
QA/QC Testing (4)	Acre	$ 500.00	30	$ 15,000.00
QA/QC Reporting (4)	Acre	$ 300.00	30	$ 9,000.00
Subtotal - Other				**$ 150,000.00**
TOTAL				**$ 2,211,546.89**

NOTES:

1 - Total cost estimates are adjusted to relect 2005 third-party dollars with estimated job specific adjustments.

2 - Foundation layer placed as part of daily/intermediate cover.

3 - 2005 Means Guide Estimated Costs. Some costs are adjusted to reflect local conditions and criteria.

4- Estimates

TABLE 2 Wasatch Regional POST-CLOSURE COST ESTIMATES SUMMARY						
LENGTH OF CLOSURE ACTIVITIES:			**30 YEARS**			
FINAL CLOSURE COSTS						**30-YEAR TOTAL**
Abandone Monitoring Wells						**$10,000**
Site Abandonment						**$75,000**
Closure Certification						**$2,000**
MAINTENANCE COSTS					**COST/YR**	
Security, fencing, gates, signs, access, etc.					$ 1,250	$ 37,500
Erosion repair, settlement repair, revegetation					$ 12,000	$ 360,000
Surface water control maintenance (run-on/run-off)					$ 4,000	$ 120,000
Storm Drainage Pipe Maintenance and Repair					$ 1,500	$ 45,000
Groundwater Drain Flow Line Maintenance					$ 2,000	$ 60,000
Leachate collection system					$ 1,000	$ 30,000
Subtotal						**$ 652,500**
MONITORING COSTS	**# OF WELLS/PTS.**	**# OF SAMPLES**	**FREQ/ YR**	**COST/ SAMPLE**	**COST/ YEAR**	
Groundwater Monitoring						
3rd Party/Sample Collection	2		2.0	$ 800	$ 3,200	
Lab Analysis		2	2.0	$ 1,500	$ 6,000	
Statistical and Reporting		2	2.0	$ 1,000	$ 4,000	
Storm Water Monitoring		1	2.0	$ 1,000	$ 2,000	
Landfill Gas Monitoring			4.0	$ 1,000	$ 4,000	
Administration Oversight					$ 20,000	
Subtotal					**$ 39,200**	**$ 1,176,000**
Total						**$ 1,915,500**

6/24/2005

Wasatch Regional
CLOSURE/POST-CLOSURE CARE COST ESTIMATE

SIZE OF CLOSURE AREA:	30 ACRES		
TOTAL CLOSURE COSTS		$	2,211,546.89
TOTAL POST-CLOSURE COSTS		$	1,915,500.00
TOTAL COST ESTIMATES:		**$**	**4,127,046.89**

NOTES:

1 - Cost estimates are adjusted to relect 2005 third-party dollars.

2 - Corrective actions are currently not occurring on-site.

6/24/2005

ALLIED WASTE SERVICES

HAND DELIVERED

JUN 22 2005
05.02154
UTAH DIVISION OF
SOLID & HAZARDOUS WASTE

June 22, 2005

Dennis R. Downs
Utah Department of Environmental Quality
Division of Solid and Hazardous Waste
288 North 1460 West
Salt Lake City, UT 84114

RE: Wasatch Regional Landfill, Inc.
Municipal Solid Waste
Landfill Permit Modification
Closure, Post-Closure Care
And Financial Assurance

Dear Mr. Downs,

We are submitting for your review the Municipal Solid Waste Landfill Permit Modification Closure, Post-Closure Care And Financial Assurance.

If you have any questions please contact me at 801-924-8485

Sincerely,

Les Lemmon

Lester Lemmon
Operations Manager
Wasatch Regional Landfill, Inc.

WASATCH REGIONAL LANDFILL, INC.

MUNICIPAL SOLID WASTE LANDFILL PERMIT MODIFICATION CLOSURE, POST-CLOSURE CARE AND FINANCIAL ASSURANCE

June 2005

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Appendix 3.1 – Estimated Closure & Post-Closure Care Costs

Section 1 - Closure Plan

This Closure Plan was developed in accordance with the Utah Administrative Code (R315-302-3). The closure of the Wasatch Regional Landfill will be completed in accordance with this plan. Closure activities will be performed in such a manner as to accomplish the following goals:

Minimize the need for further maintenance;
Minimize or eliminate threats to human health and the environment from post closure escape of solid waste constituents, leachate, landfill gases, contaminated run-off or waste decomposition products to the ground, groundwater, surface water, or the atmosphere and;
Adequately prepare the facility for the post-closure period.

This Closure Plan and any future modifications or changes to this plan will be maintained as part of the landfill's operating record.

Elements of Closure

Closure may include final grading and contouring, liner placement, seeding, or placement of stone mulch. Storm water design and control will be part of closure activities. Final closure construction will typically be initiated within one year after a landfill area reaches final grade. Closure will occur in small phases and may include any combination of side slope or top area. It is anticipated closure may occur every year. Prior to proceeding with any closure activities, Design Drawings and a QA/QC Plan will be submitted to the Executive Secretary for review and approval of the proposed activities.

Closure Schedule

Wasatch Regional Landfill will notify the Executive Secretary of the intent to implement the closure plan at least 60 days prior to closure activities. This notification will provide details on which area will be closed and how the final cover will be constructed. It will also include a QA/QC document and engineered construction drawings.

Within two years after a landfill area is to final grade. Wasatch Regional Landfill will implement the closure plan, and will complete closure activities within 180 constructions days. Following the completion of final closure activities, Wasatch will submit within 30 days to the Executive Secretary a set of as-built drawings of final closure construction signed by a professional engineer registered in the State of Utah. Wasatch will also provide certification of the compliance of each phase of closure construction with the approved closure plan. A representative of Wasatch and a professional engineer registered in the State of Utah will sign the certification.

Closure Design

The current final cover design concept and engineering report includes graded intermediate soil cover material, GCL, textured 60 mil HDPE, 18 inches of soil cover above the liner and either 6 inches of top soil followed by seeding or 21 inches of soil cover followed by 3 inches of a stone mulch.

It is anticipated an Alternative Soil Cover Design application will be submitted to the Division for review and approval sometime during 2006.

Final Inspection

Following the completion of closure activities, a final report will be prepared and certified by an engineer registered in the State of Utah. The report will present laboratory and field test data that support the conformance of the final cover installation and closure activities with the Utah Solid Saste regulations and the approved Closure Plan. The report will also include facility closure plan sheets signed by a professional engineer registered in the state of Utah that represent the final, as-built closure construction. The Executive Secretary will be notified of the completion of closure activities and arrangements will be made for a final inspection by DEQ. Following final approval by DEQ, the post-closure plan will be initiated pursuant to the approved Post-Closure Plan.

Section 2 – Post-Closure Care Plan

Post-Closure Care Plan

This Post-Closure Plan has been developed in accordance with UAC R315-302-3, and provides for post-closure care and maintenance of the Wasatch Regional Landfill. All post-closure maintenance and monitoring will be performed in accordance with this plan.

Elements of Post Closure

Post Closure will include maintenance and monitoring of gases, land and water for 30 years or as long as the Executive Secretary determines necessary for the facility to become stabilized and to protect human health and the environment. Post Closure activities will include: leachate management, filling areas of differential settlement, erosion control, storm water management, gas collection and control, groundwater sampling and management, air monitoring and reporting, site security and site management.

Post-closure Schedule

The Post-closure maintenance period will begin immediately following the completion of all landfill unit closure activities. Post-closure activities will continue for a period of thirty years or a period established by the Executive Secretary. If, during the post-closure period, monitoring activities indicate that the site has stabilized and does not pose a threat to human health or the environment. Wasatch may petition the Executive Secretary for a decrease in the length of the post-closure monitoring period. Following completion of the post-closure monitoring period as established by the Executive Secretary, Wasatch will submit to the Executive Secretary a certification, signed by an authorized representative of Wasatch and a professional engineer registered in the State of Utah, which states why post-closure monitoring activities are no longer necessary. After obtaining final approval from the Executive Secretary, post-closure monitoring activities will be discontinued. Any modifications to the post-closure plan will be submitted to the Executive Secretary for approval at least 6 months prior to the implementation of the post-closure plan.

Monitoring

Monitoring activities will include groundwater, landfill gas, leachate, storm water as necessary and any air quality items as required

If continued monitoring at the facility indicates that the waste mass has stabilized and does not pose a threat to human health or the environment, the owner or operator may petition the Executive Secretary for a decrease in the length of the post-closure monitoring period. Records for all monitoring activities will be stored at the Wasatch Regional Landfill.

Maintenance Activities

During the post-closure period, personnel from Wasatch will inspect: the final cover for differential settlement and erosion, the storm water channels and drainage systems to assure they are clean and working properly, the site boundary security fences. In addition, all groundwater and landfill gas monitoring equipment will be inspected according to the manufacturers recommendation. If the inspection indicates that there is a need for repairs, the appropriate sub-contractor will be immediately contacted. Repairs will be completed as soon as possible following each inspection in order to maintain the effectiveness of the monitoring equipment.

Planned Use of Property

Currently, there are no planned uses of the property during the post closure period.

Section 3 - Cost Estimates For Closure & Post-Closure

Costs associated with the closure and post-closure period have been calculated for the initial permit term of five years. The cost estimates have been based on the most expensive cost to close the largest area of the disposal facility requiring closure during the permit. The largest open area requiring closure during the five year permit period is 30 acres which occurs during year 3. After year 3 the first construction phase will be partially closed leaving a maximum area of 30 acres requiring closure through year 5. Additional closure costs that will occur during year 3 that will not be required at year 5 include downspout piping at the southeast corner of the landfill area and the rip rap drainage channel along the south side and extending around the west side of the landfill area. These estimates are based on 2005 construction costs, 2005 Means Guide, and estimated engineering and surveying costs. The estimated closure and post-closure maintenance costs for the first five years of operation are presented in Appendix 3.1. A financial assurance mechanism will be submitted to the Executive Secretary for approved and become effective prior to operation and initial receipt of waste at the facility.

The specific quantities of materials used in calculating the closure/post-closure costs were measured from design plans (provided in the permit drawings) assuming a constructed landfill area of 30 acres. The projected post-closure costs were calculated on the assumption that the integrity of the final cover would be inspected annually, landfill gas would be monitored quarterly, ground water would be monitored semiannually, and that general facility maintenance would be ongoing. Final closure and post-closure costs will be evaluated and adjusted annually. These estimates may change as a result of permit modifications, regulatory changes, operational changes, or changes in the closure total acreage. If corrective action is anticipated during the post-closure period, additional closure estimates and financial assurance will be provided.

TABLE 1 Wasatch Regional FINAL YEAR 3 CLOSURE COST ESTIMATES SUMMARY				
SIZE OF CLOSURE AREA:	**30.0 ACRES**			
CLOSURE COSTS	**MEASURE**	**UNIT COST**	**QUANTITY**	**TOTAL**
Supply & Placement of Closure Cap				
General Contractor Mobilization/Demobilization [1]	Lump Sum	$ 25,000.00	1	$ 25,000.00
Liner Contractor Mobilization/Demobilization [1]	Lump Sum	$ 5,000.00	1	$ 5,000.00
GCL [1]	Acre	$ 17,563.00	30	$ 526,890.00
60 Mil HDPE Textured [1]	Acre	$ 20,440.00	30	$ 613,200.00
Freight and Material Taxes [1]	Included in $/Acre			
Soil Cover (21") [1]	Acre	$ 13,419.00	30	$ 402,570.00
Grading of Waste/Surface Preparation [3]	Acre	$ 1,000.00	30	$ 30,000.00
Surveying [4]	Acre	$ 500.00	30	$ 15,000.00
Stone Mulch [1]	Acre	$ 940.00	30	$ 28,200.00
Subtotal				**$ 1,645,860.00**
Stormwater Controls				
Channel Excavations [3]	LF	$ 2.00	2500	$ 5,000.00
Riprap Channel Granular Filter (run-on control) [3]	CY	$ 10.00	1400	$ 14,000.00
Riprap Channel Riprap (run-on control) [3]	CY	$ 50.00	2100	$ 105,000.00
Downchute Pipe [3]	LF	$ 57.50	400	$ 23,000.00
Inlet Boxes [3]	EA	$ 2,500.00	2	$ 5,000.00
Subtotal				**$ 152,000.00**
Other: (List)				
Engineering Site Evaluation [4]	LS	$ 10,000.00	1	$ 10,000.00
Design, Specification & CQA/CQC Manual [4]	LS	$ 50,000.00	1	$ 50,000.00
Project Mgmt. & QA/QC, Oversight [4]	Acre	$ 2,500.00	30	$ 75,000.00
QA/QC Testing [4]	Acre	$ 500.00	30	$ 15,000.00
QA/QC Reporting [4]	Acre	$ 300.00	30	$ 9,000.00
Subtotal - Other				**$ 150,000.00**
TOTAL				**$ 1,947,860.00**

NOTES:

1 - Total cost estimates are adjusted to relect 2005 third-party dollars with estimated job specific adjustments.

2 - Foundation layer placed as part of daily/intermediate cover.

3 - 2005 Means Guide Estimated Costs. Some costs are adjusted to reflect local conditions and criteria.

4- Estimates

TABLE 2
Wasatch Regional
POST-CLOSURE COST ESTIMATES SUMMARY

LENGTH OF CLOSURE ACTIVITIES: 30 YEARS

FINAL CLOSURE COSTS						30-YEAR TOTAL
Closure Certification						$ 2,000
MAINTENANCE COSTS					**COST/YR**	
Security, fencing, gates, signs, access, etc.					$ 1,250	$ 37,500
Erosion repair, settlement repair, revegetation					$ 12,000	$ 360,000
Surface water control maintenance (run-on/run-off)					$ 4,000	$ 120,000
Storm Drainage Pipe Maintenance and Repair					$ 1,500	$ 45,000
Leachate collection system					$ 1,000	$ 30,000
Subtotal						**$ 592,500**
MONITORING COSTS	**# OF WELLS/PTS.**	**# OF SAMPLES**	**FREQ/ YR**	**COST/ SAMPLE**	**COST/ YEAR**	
Groundwater Monitoring						
3rd Party/Sample Collection	2		2.0	$ 800	$ 3,200	
Lab Analysis		2	2.0	$ 1,500	$ 6,000	
Statistical and Reporting		2	2.0	$ 1,000	$ 4,000	
Storm Water Monitoring		1	2.0	$ 1,000	$ 2,000	
Landfill Gas Monitoring			4.0	$ 1,000	$ 4,000	
Administration Oversight					$ 20,000	
Subtotal					**$ 39,200**	**$ 1,176,000**
Total						**$ 1,770,500**

Wasatch Regional CLOSURE/POST-CLOSURE CARE COST ESTIMATE		
SIZE OF CLOSURE AREA: 30 ACRES		
TOTAL CLOSURE COSTS	**$**	1,947,860.00
TOTAL POST-CLOSURE COSTS	**$**	1,770,500.00
TOTAL COST ESTIMATES:	**$**	**3,718,360.00**

NOTES:

1 - Cost estimates are adjusted to relect 2005 third-party dollars.

2 - Corrective actions are currently not occurring on-site.

36514

HAND DELIVERED

JUN 0 8 2005
05.02014
UTAH DIVISION OF
SOLID & HAZARDOUS WASTE

ALLIED WASTE SERVICES

June 8, 2005

Dennis R. Downs
Utah Department of Environmental Quality
Division of Solid and Hazardous Waste
288 North 1460 West
Salt Lake City, UT 84114

RE: Wasatch Regional Landfill, Inc.
Response to Class V Landfill Permit Modification Review
Request for Additional Information #1 (April 22, 2005)

Dear Mr. Downs,

We are submitting for your review the response to the Class V Landfill Permit Modification Review Request for Additional Information #1 (April 22, 2005) for the Wasatch Regional Landfill, Inc. facility as prepared by Hansen, Allen & Luce, Inc.

If you have any questions please contact me at 801-924-8485

Sincerely,

Lester Lemmon
Operations Manager
Wasatch Regional Landfill, Inc.

HANSEN
ALLEN
& LUCE INC
ENGINEERS

June 7, 2005

Mr. Dennis R. Downs
Executive Secretary
Utah Department of Environmental Quality
Division of Solid and Hazardous Waste
288 North 1460 West
P.O. Box 144880
Salt Lake City, Utah 84114-4880

Re: Wasatch Regional Landfill, Inc.
Response to Class V Landfill Permit Modification Review
Request for Additional Information #1 (April 22, 2005)

Dear Mr. Downs:

The attached is the response to the Class V Landfill Permit Modification Review Request for Additional Information #1 (April 22, 2005) for the Wasatch Regional Landfill, Inc. Facility. This response has been prepared to provide the additional information requested for each comment resulting from your review of the requested design permit modification as provided in the "Municipal Solid Waste Landfill Permit Modification Design Engineering Report" dated December 2004 prepared by Hansen, Allen & Luce, Inc. Revised pages of the Design Engineering Report, additional calculations and requested documents are also provided.

Please contact us with any questions or comments you may have regarding the additional information provided herewith.

Sincerely,
HANSEN, ALLEN & LUCE, INC.

Kent C. Staheli, P.E.
Principal

cc. Kory Coleman, Vice President - Wasatch Regional Landfill, Inc.
Darin Olson, Environmentatl Manager - Wasatch Regional Landfill, Inc.
Kirk Treese, Manager - Wasatch Regional Landfill, Inc.
Lester Lemmon, Operations Manager - Wasatch Regional Landfill, Inc.
Kirk Treese, Manager - Wasat

GENERAL COMMENTS

As part of the permit review process, this modification will be made available for public comment. When drafting changes to the application, it should be kept in mind that this document might be viewed by individuals without technical knowledge of landfill design or operation.

- Please keep in mind the broad audience of individuals that will review the application.

- Note: Reference to the Sections of the *Utah Solid Waste Permitting and Management Rules - R315-301 through 320* will simply be referred to by the Section number, for example R315-302-2.

The submitted modification is a major modification as defined by SectionR315-311-2. Class V permit modifications are subject to a fee fo $70 per hour of review time as authorized by the Appropriations Act SB#1.

- Once the modification is determined complete a 30-day public comment period will be initiated.

CHAPTER II - GROUNDWATER

Recharge Estimates
Page II-3 states:

> *Stephens (1974) indicated in Technical Publication No. 42 (TP42) that the average percent of precipitation contributing to groundwater recharge for the periphery of the Northern Great Salt Lake Desert, which include the Lakeside Mountains, is 3%. Specific recharge was not addressed for the Lakeside Mountains in TP-42 and a recharge rate of 5% of precipitation for this area was assumed in the model to be conservative.*

Comment #1

> As a reference, please include all or the relevant portion of TP-42 to document the appropriate recharge rate.

✓ **Response #1**

> Submitted with this response is a revised set of calculations for the ground water modeling effort to replace the calculations provided in Appendix C of the "Wasatch Regional Landfill Municipal Solid Waste Landfill Permit Modification - Design Engineering Report" dated December 2004. These calculations include the title page and the relevant figures and table from TP-42 from which the 3% recharge estimate was obtained.

Hydraulic Conductivity and Model Calibration
Page II-4 states with support of Figure II-4:

> *The computed groundwater levels were less than 2 feet above the observed levels in three of the six observation points within the southern half of the facility. The computed levels were less than 2.2 feet below the observed levels at the other three southern observation points. Therefore, the computed groundwater levels in the southern half of the facility are considered to be a reasonable representation of actual groundwater elevations.*

Comment #2

The Rules require a minimum of a five-foot separation between the lowest liner and the historic high ground water level. The ground water model for the southern half of the landfill over predicted the water levels by 1.8 feet and under predicted water levels by as much as 2.2 feet. To account for the potential model error, the provided soil compaction calculation, and the required five-foot separation, the minimum separation between the liner and historic high groundwater needs to be 8.2 feet. This separation needs to be documented in the text and drawings contained in the modification.

Response #2

The ground water model has been revised to provide a drain trench located closer to the east side of the landfill area. Moving the trench closer to the landfill area and providing a bottom elevation for the trench of 4227 results in minimum separation of 9.5 feet between the project high ground water and the lowest point in the bottom liner system for all phases of the landfill. Calculations are attached to include with the other calculations in Appendix C of the Design Engineering Report.

The first paragraph following the discussion titled "Drain Trench" on Page II-5 of the Design Engineering Report has been modified to present the results of the modified ground water model and drain trench design. Also the calculations presented in Appendix D - Floor Elevations have been modified and are included herewith to replace the original calculations.

Projected High Groundwater Level
Page II-4 declares:

> *Maximum groundwater levels were computed by inserting the recharge data from 1980 to 1983 and the Great Salt Lake elevation from 1985 into the calibrated model.*

Comment #3

The modification needs to state the specific elevation above MSL used in the groundwater model.

Response #3

The highest recorded Great Salt Lake level is at an elevation of 4211.85 in 1985. An elevation of 4212 was used for modeling purposes. The only other recorded Great Salt Lake level that was near 4212 occurred in about 1870 and that recorded level was also just below 4212. The text in Chapter II was modified to provide the specific maximum Great Salt Lake level as used in the model.

NOTE: Attached are revised Figures II-6 and II-7 representing projected ground water contours with the modified drain trench location to replace the original figures.

Liner System

Page III-4 explains the interior slopes will have a 2H: IV slope

Comment #4

It is not clear if the entire liner will immediately be covered with protective soil. The modification needs to clearly state when protective cover will be applied to the liner and GCL. If a protective cover is not placed immediately after construction, the modification needs to include documentation that demonstrates the GCL will not prematurely hydrate and that the integrity of the liner will be maintained.

Response #4

In order to provide for the desired stability of the protective soil cover and liner system on the interior cell slopes and to minimize stresses in the liner system, the protective soil cover will be placed on the slopes in two phases. Each phase will consist of soil cover placement to a vertical height of approximately 10 feet. The lower 10 feet will be placed on the interior slopes at the time the protective soil cover is placed on the landfill floor area during construction of landfill phases or sub-phase. The final 10 feet (or the remaining slope area) will be placed when the first lift of waste is placed adjacent and above the lower 10 feet of the slope area.

The GCL materials that will be placed on the interior side slopes will consist of needle punch reinforced GCL materials. These GCL materials are typically manufactured with a bentonite moisture content around 20%. Test results presented by the U.S. Environmental Protection Agency "Report of 1995 Workshop on Geosynthetic Clay Liners", EPA/600/R-966/149, dated June 1996 show that hydration will occur in GCL materials in direct contact with prepared soil subgrades. The prepared subgrade materials will be placed and compacted at a maximum moisture content of 4% above OMC. Therefore, the moisture content of the GCL materials after hydration from moisture contained within the subgrade soils is expected to be below 100% OMC.

CETCO's Technical Services conducted laboratory testing on bentonite material and on their needle punch reinforced GCL to determine the swelling properties of the bentonite material under various confining pressures and to determine an approximate confining strength of the needle punch reinforced GCL. During the tests, the test vessels were filled with de-ionized water to allow the bentonite material to freely absorb water. Results from the laboratory tests show that the bentomat (needle punch reinforced) GCL provided a confining strength equivalent to a 10.7 Kpa overburden load which is equivalent to about 500 mm (20 inches) of overburden soil. The tests on the bentonite were conducted under conditions that allowed for free absorption of water within the test apparatus resulting in complete hydration of the material.

Test data provided by the USEPA show that actual conditions will limit absorption of water within the GCL to provide a moisture content within the bentonite of less than 100%. Since the top surfaces of the embankments are designed to drain storm water away from the liner systems, there should be no added source of water for GCL hydration other than the moisture used for construction. Based on the test results and the limited hydration that will occur in the GCL, we feel that the strength properties of the needle punch reinforced GCL will provide confining strengths similar to the confining pressures that will result from placement of the protective cover material. Reports and test data conducted by USEPA and by CETCO are provided with this response.

All geomembrane materials left exposed for subsequent placement of protective soil cover will be inspected for damaged areas and repaired prior to placement of additional protective soil cover materials.

Groundwater Monitoring Wells
R315-308-2 requires any point along the unit boundary shall be within 500 feet of a ground water monitoring well. A portion of the northern unit boundary of phase 11 is not within 500 feet of a monitoring well.

Comment #5

An additional ground water monitoring well needs to be placed along the northern boundary of phase 11.

Response #5

Sheets 3 and 4 of the drawings have been modified to show an additional monitoring well located approximately 500 feet to the west of the interior northeast corner of the landfill (along the north side of Phase 11).

Phage III-11 declares that one monitoring well upgradient and two downgradient monitoring wells have been installed. The following conditions are part of the issued Class V permit:

The Permittees shall modify the Ground Water Monitoring Plan to reflect the installation of the groundwater monitoring wells. The modified Ground Water Monitoring Plan shall be submitted to the Executive Secretary for review. The modified Ground Water Monitoring Plan must be approved by the Executive Secretary prior to receipt of waste at the landfill. The modified Ground Water Monitoring Plan must include surveyed as-builts, well logs, detailed drawings and maps for all the groundwater monitoring wells, and any necessary changes to the ground water QA/QC Plan, sampling procedures, and *statistical methods.*

Comment #6

The changes to the Groundwater Monitoring Plan need to be submitted for review.

Response #6

A revised Groundwater Monitoring Plan has been prepared by The Carel Corporation located in Keller, Texas and submitted by Wasatch Regional Landfill, Inc. to provide for an intra-well sampling and analysis program. Intra-well sampling is requested due to the inability to construct a reliable up-gradient monitoring well at the facility.

Measurements of the water levels in the existing boring that was anticipated to provide for an up-gradient monitoring well constructed west of the Phase 1 area show that reliable sampling will not be possible. This monitoring well was drilled through approximately 143 feet of gravel sediments and approximately 30 feet additionally into the underlying bedrock for a total of 173 feet.

Groundwater was not observed during drilling, however, the boring was left open for a couple of days and checked with a water level indicator probe for the presence of ground water. Ground water was measured at about 154 feet (about 11 feet below the bedrock surface). We feel that the presence of groundwater was not observed during drilling because of the slow recharge through the bedrock and the air lifting of drill cuttings dried the cuttings prior to reaching ground surface. PVC casing and screens were installed to provide screening that extended to about 158 feet and a blank chamber extending approximately 5 feet below the bottom of the screen.

A pneumatic pump was installed in the well on March 24, 2005 to purge the well. During installation and purging of the well, it was observed that the recovery rate within the well was extremely slow. The discharge rate for the pump was set to less than 0.1 liter per minute and we were only able to obtain what would equate to about one well volume from the ground water surface to the location of the pump at the bottom of the screen. We returned to the well several hours later and no recovery had taken place in the well. Since the well is in the bedrock, the ground water level is below the surface of the bedrock, and the recharge rate is so slow, we feel that consistently quality samples will not be possible. Additionally, we feel that all potential monitoring well locations west of the landfill will yield similar results.

CHAPTER IV - LANDFILL CLOSURE DESIGN

Page IV-1 states:

A final cover system consisting of 60-mil HDPE textured geomembrane and 2 feet of cover material is placed above the waste mound.

Page 4 of Appendix B includes the design of the final cover system. The design does not include a GCL.

R315-303-3(4)(A) states:

In no case shall the cover of the final lifts be more permeable than the bottom liner system or natural subsoils present in the unit.

Comment #7

Since the bottom liner consists of a 60-mil HDPE and a GCL, the final cover design must include a system that is no more permeable. Accordingly, the standard design of the final cover needs to include a final cover design that incorporates a product equivalent to the bottom GCL. If an alternative design is proposed, it must include a detailed demonstration to show that it achieves the equivalent reduction in infiltration as the standard final cover system.

Response #7

Figure 6-3a of the "Solid Waste Disposal Facility Criteria - Technical Manual" prepared by the US Environmental Protection Agency (USEPA) as EPA530-R-93-017 dated November 1993 requires only that an 18 inch thick infiltration layer meeting a permeability of $1 \times 10^{-5}\ {}^{cm}/_{sec}$ overlain by a flexible membrane liner be constructed where the bottom liner system consists of a 2-foot thick compacted soil meeting a permeability of $1 \times 10^{-7}\ {}^{cm}/_{sec}$ overlain by a flexible membrane liner. It is our understanding that this criteria was verified with the USEPA.

Although it is the position of Wasatch Regional Landfill, Inc. that the minimum requirements would be met by following the requirements presented in the EPA technical manual, a GCL has been added to the closure details as requested by the DSHW in meeting with the DSHW interpretation of criteria required by 40CFR Part 258.

CHAPTER V - STORMWATER MANAGEMENT

Page V-2 discusses the Areal Reduction Factor based on the Salt Lake City Hydrology Manual. However, it is unclear how the ARF results were used in the calculation. Does the ARF of 0.96 mean the storm event rainfall amount was reduced by 4%?

Comment #8

Please provide a discussion of how the ARF was used in the calculations.

Response #8

The Areal Reduction Factor (ARF) is applied to the precipitation value for each of the sub-basins to generate peak design flow rates. Therefore, the precipitation value is reduced by 4% to provide a precipitation value of 96% of the values obtained from the NOAA atlas.

APPENDIX A

Sheet 8 shows a typical embankment cross section. No slope detail is provided to ensure drainage away from the disposal cell.

Comment #9

The degree of slope away from the waste cell needs to be included in the drawings.

The modification does not include the drawing to show how the final cover will be tied into the bottom liner.

Response #9

Sheet 8 has been modified to show a cross slope of 1.0% minimum.

Comment #10

The modification needs to include a typical cross section of showing the final cover liner and bottom liner tie in.

Sheet 10 Cell Phase Division Berm shows no anchor for the future cell liner.

Response #10

Cross-section 8 on Sheet 4 has been added to show the tie-in.

Comment #11

The modification needs to demonstrate that the HDPE weld alone is adequate to maintain the integrity of the liner.

Response #11

Welds are tested to be stronger than the geomembrane sheet and all destructive testing during construction requires that the strengths of the seams exceed that of the sheet material. The tie-in seam is located on top of the phase division berms between two adjacent phases and will include a continuous seam along the length of the tie-in. We recommend destructive tests be conducted every 500 feet along the tie-in seam to demonstrate strength acceptance.

Stresses during construction and operation should be minimal since the berms are sufficiently low that they will be covered with protective soil cover during construction and will be covered with waste material with the first lift of waste placement. The materials will, therefore, be self supporting and will not provide stresses beyond the strength of the geomembrane and the tie-in seam.

APPENDIX B

Page one in the Floor Elevation section in Appendix B contains a table, which includes the "Separation Between Projected High Ground Water". The calculations in the table are not clearly explained.

Comment #12

Please provide additional explanation to show how the separation is calculated on page one of Floor elevations.

Response #12

The table has been corrected and updated to provide clarity and to reflect the conditions of the modified location for the groundwater drain trench.

Page 21 states:

We recommend that the strength of the proposed synthetic materials and the underlying soils be verified prior to construction.

Comment #13

To implement this proposal, the QA/QC plan will need to include the recommended testing.

Response #13

Page 21 states "The integrity and desired factor of safety may be achieved on the 2:1 slopes by placing the soil protective cover in 10-foot vertical stages or by verifying that the interface strength between the GCL and underlying soil on the slope is greater than we have assumed. The literature indicates that a higher strength will most likely apply. We recommend that the strength of the proposed synthetic materials and the underlying soils be verified prior to construction." The design presented in the drawings shows the option of placing the soil protective cover in 10-foot vertical stages so that the integrity and desired factor of safety is achieved without the additional testing and verification.

Page 14 states:

This acceleration was adjusted for the stability analysis as recommended in the DMG Special Publication 117 "Guidelines for Analyzing and Mitigating Landslide Hazards in California." Using this document, an acceleration of 0.092g was used for the stability calculations assuming a threshold of 15 cm displacement.

Comment #14

The staff has used the RCRA subtitle D (258) *Seismic Design Guidance for Municipal Solid Waste Facility*. However, the staff is not familiar with Publication 117. A copy of the publication needs to be included in the Modification with a discussion of how it was applied in the model.

Response #14

Attached is the response provided by Applied Geotechnical Engineering Consultants (AGEC) and a copy of the requested Publication 117.

Page 15 states:

The testing consisted of penetration resistances, unconfined compression strength test, triaxial shear test and direct shear test conducted on undisturbed and remolded soils samples. Based on these results, previous testing by others and our judgment, strength parameters for each material were selected.

Comment #15

Specific reference to test results and supporting data need to be provided to support each one of the selected parameters. As one example, strength parameters provided on page 15 shows the unit weight for waste as 120 pcf. The Class V permit application used a unit weight of 72.6 pcf for waste. The modification needs to include the justification for using another number.

Response #15

Attached is the response provided by AGEC including a discussion of the parameters assumed and how those parameters compare with laboratory test results. In all cases the parameters provided for a conservative design.

There are also additional areas where parameters may vary from those in the original permit application. Some of these parameters may include using a unit weight of 80 pounds per cubic foot for waste material in determining loadings on the geonet component of the leachate collection system. The unit weight assumed is slightly higher resulting in a heavier loading and more conservative results for the design. In each case, where parameters have been selected, Hansen, Allen & Luce, Inc. and AGEC have attempted to make assumptions that will result in a conservative design.

Note: Financial Assurance for the landfill will need to be provided and approved prior to acceptance of waste. As per R315-309-2(3)(a) the closure cost estimate shall be based on the most expensive cost to close the largest area of the disposal facility ever requiring a final cover at any time during the active life (Permit Life - 5 years) in accordance with the closure plan.....

Response to Note

Financial assurance estimates will be provided in a separate letter. We understand that the financial assurance amount will be required to start the 30 day public comment period and that the financial assurance mechanism is required to be in place prior to receipt of waste materials.

RECEIVED

APR 22 2005
05.01590
UTAH DIVISION OF
SOLID & HAZARDOUS WASTE

36274

ALLIED WASTE

675 South Gladiola St.
Salt Lake city, UT 84104

April 14, 2005

Mr. Dennis Downs
Executive Secretary
Utah Department of Environmental Quality
Division of Solid and Hazardous Waste
P.O. Box 144880
Salt Lake City, Utah 84114-4880

Re: Ownership Change of Wasatch Regional Solid Waste Management Corp., & Authorization for Design Permit Modification and Quality Control Plan

Dear Mr. Downs:

This letter is to inform the Utah Division of Solid and Hazardous Waste (DSHW) of the purchase of Wasatch Regional Solid Waste Management Corp by Wasatch Regional Landfill, Inc., which is a corporation solely owned by Allied Waste Company.

This letter also authorizes submittal and review of the following which have been prepared by Hansen, Allen & Luce, Inc.:

> AWasatch Regional Landfill Municipal Landfill Permit Modification Design Engineering Report@, dated December 2004.

> AWasatch Regional Landfill, Inc. 2005 Construction Quality Assurance Construction Quality Control (CQA/CQC) Plan for Landfill Construction,@ dated April 2005.

Designated authorized representative for Wasatch Regional Landfill Inc., as per R315-310-2(4), are::

> Mr. Kory Coleman, Vice President
> Mr. Lester Lemon, Operations Manager

mailing address at:

> 675 South Gladiola
> Salt Lake City, Utah 84104
> (801) 972-4234

and:

Mr. Kirk Treese, General Manager
Mr. Darin Olson, Environmental Manager

mailing address at:

1111 West Highway 123
P.O. Box 69
East Carbon, Utah 84520

Phone No. (435) 888-4418
Fax (435) 888-0407

If you have any questions relating to the above information, please contact us at the above indicated phone number or address.

Sincerely,

WASATCH REGIONAL LANDFILL, INC.

Kory Coleman
Vice President

cc Mr. Jeff R. Coombs, Tooele County Environmental Health Supervisor
Mr. Jim Lawrence, P.E., Tooele County Engineer
Mr. Barry Formo, Tooele County Building Official
Ms. Nicole Cline, Tooele County Planning

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ENGINEERS

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DEC 2 7 2004
04.04304
UTAH DIVISION OF
SOLID & HAZARDOUS WASTE

December 27, 2004

Mr. Ralph Bohn
Utah Department of Environmental Quality
Division of Solid and Hazardous Waste
288 North 1460 West
P.O. Box 144880
Salt Lake City, Utah 84114-4880

Re: Wasatch Regional Solid Waste Landfill Facility
Design Engineering Report for Design Permit Modification

Dear Mr. Bohn:

As requested by Mr. Darin Olson of ECDC Environmental, we are transmitting herewith are two copies of the Design Engineering Report for a Design Permit Modification request for the above referenced project. The report is submitted for your review and permit modification approval.

Please contact us with any questions or comments you may have regarding the information contained herein.

Sincerely,
HANSEN, ALLEN & LUCE, INC.

Kent C. Staheli, P.E.
Principal

cc. Darin Olson, ECDC Environmental L.C.

WASATCH REGIONAL LANDFILL

MUNICIPAL SOLID WASTE LANDFILL PERMIT MODIFICATION DESIGN ENGINEERING REPORT

REGISTERED PROFESSIONAL ENGINEER
12-20-04
NO. 169141
KENT C. STAHELI
STATE OF UTAH

Project Engineer

Prepared by:

HANSEN, ALLEN & LUCE, INC
Consulting Engineers
6771 South 900 East
Midvale, Utah 84047
(801) 566-5599

December 2004

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CHAPTER I

INTRODUCTION

Hansen, Allen & Luce, Inc. was retained to provide engineering services for a proposed design permit modification at the Wasatch RegionalLandfill to be located west of the Great Salt Lake within Sections 32, 33 and 34 of Township 2 North, Range 8 West, Salt Lake Base and Meridian and within sections 3 and 4 of Township 1 North, Range 8 West, Salt Lake Base and Meridian. The facility property and adjacent properties to the east, west and south are currently owned by the Utah State Institutional Trust Lands Administration (SITLA).

The proposed permit modification will include modifying the current permitted design. Design modifications include:

1. Providing a ground water interceptor trench to isolate the facility ground water levels from fluctuations in Great Salt Lake levels and to provide a drain for the interceptor trench.
2. Reducing the landfill operating area.
3. Locating the landfill operating area in the western part of the facility (adjacent to the west mountains, or Lakeside Mountains) to allow borrow materials to be obtained from the eastern part of the property. This configuration provides a design with a closer balance of required cut and fill soil quantities.
4. Modifying the configuration of the leachate collection and removal system and the floor elevations.
5. Reducing the height and configuration of the waste mound and final closure cap.
6. Moving the location of the proposed ground water monitoring wells.
7. Modifying the storm water run-on control system.

Locations and configurations of other on-site facilities to support landfill operations were modified to provide a general concept regarding the types of facilities needed. These facilities include access roads, access control fencing and gates, truck scales, office trailer or building, maintenance building, leachate management pond(s) to be used after closure, and parking areas. The locations, sizes and configurations of these facilities are not critical to the design requirements associated with the landfill and its closure. Therefore, it is understood that the types and locations of proposed support facilities may be modified from those presented.

The facility is in the permitting process as a Class I Landfill site with a future request to modify the permit to a Class V Landfill. The design provided herein is consistent with the standards of design required by the Utah Administrative Code 315 for Class I and Class V Landfills and with EPA 40CFR, Title 40, Part 258 Criteria for Municipal Solid Waste Landfills. This report provides detailed information regarding groundwater, landfill design, landfill closure design, and storm water management.

CHAPTER II

GROUNDWATER

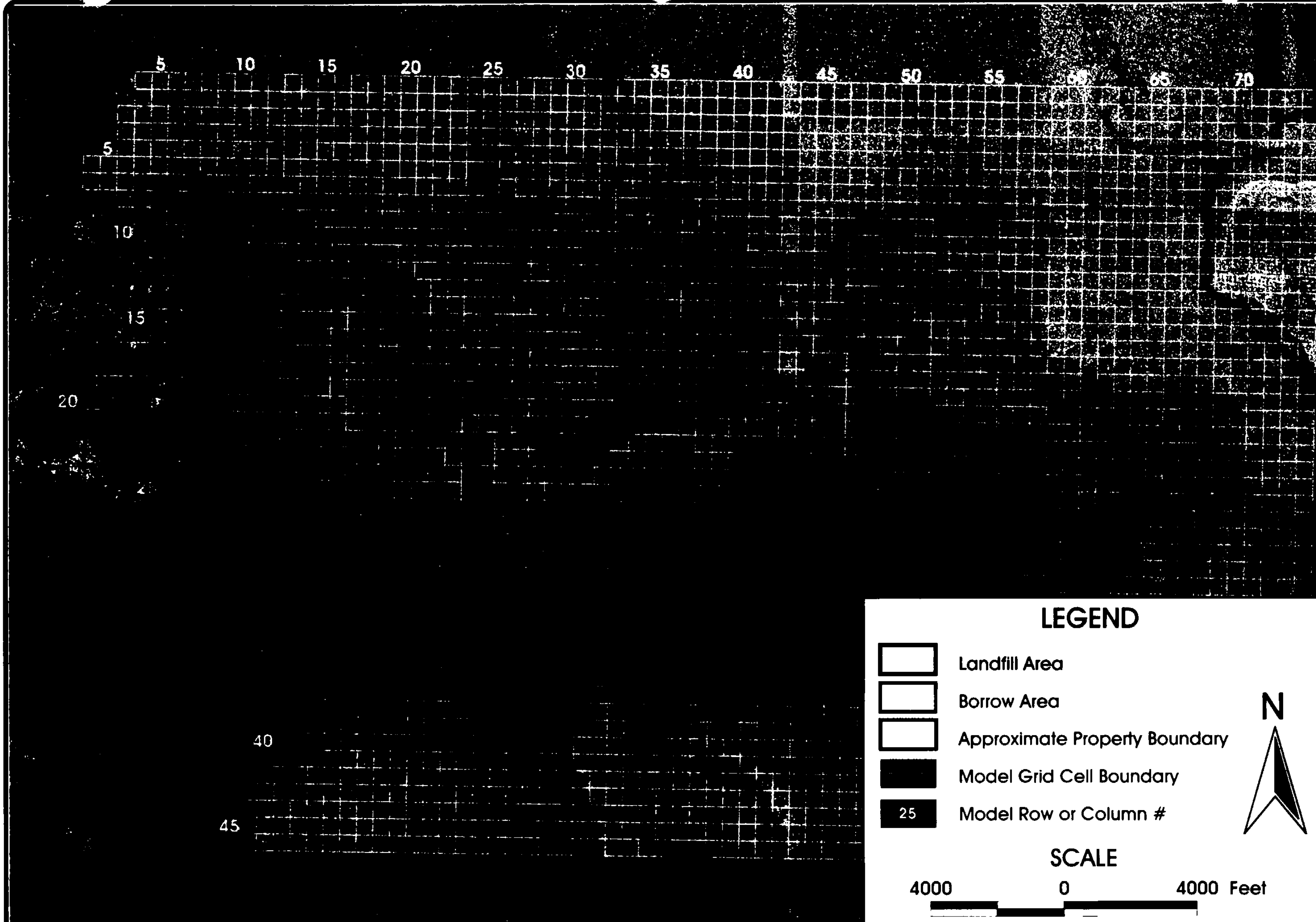
PROJECTED FUTURE GROUNDWATER CONDITIONS

Due to the lack of historical groundwater level measurements, a groundwater model of the unconsolidated aquifer in the vicinity of the Wasatch Regional Landfill was created in order to estimate maximum future groundwater conditions. MODFLOW, a modular, three dimensional, finite difference groundwater model developed by the US Geological Survey (McDonald and Harbaugh, 1988), was used to simulate groundwater conditions in the area of the landfill. MODFLOW uses a block centered grid to define the aquifer on a node by node basis. Information required by MODFLOW includes aquifer top and bottom elevations, aquifer properties such as hydraulic conductivity, aerial sources and sinks such as recharge and evapotranspiration, point sources and sinks such as wells and drains, and other boundary conditions such as general head or fixed head boundaries.

Using a steady state simulation, the model was calibrated to measured groundwater levels below the landfill site (obtained in 2003 from borehole investigations performed by Kleinfelder) by adjusting hydraulic conductivity values across the model. Precipitation and Great Salt Lake elevation data from 2000 to 2003 also were used for the calibration. Estimation of the maximum anticipated groundwater levels was accomplished by entering maximum precipitation data from 1980 to 1983 and the maximum historical Great Salt Lake elevation from 1985 into the calibrated model and then running a steady state simulation. The steady state assumption in MODFLOW results in predicted groundwater levels assuming the input conditions remained constant until the model inflow and outflow are balanced. Therefore, inputting the maximum Great Salt Lake Levels and maximum precipitation in a steady state model results in computed groundwater levels assuming these conditions persisted forever. Development of the MODFLOW model is described below and is included in Appendix C.

Study Area and Model Discretization

The Landfill site will be located west of the railroad and at the base of the Lakeside Mountains in Sections 33 and 34, Township 2 North, Range 8 West and in Sections 3 and 4, Township 1 North, Range 8 West, Salt Lake Base and Meridian (SLB&M). In order to define the MODFLOW model, a coordinate system was established running parallel with section lines, with the northeast corner of Section 28, Township 2 North, Range 8 West, SLB&M coinciding with the point x=5,000 feet and y=23,000 feet in the coordinate system. The x-axis increases to the east and the y-axis increases to the north. The model grid contains 46 rows and 74 columns consisting of square cells with 500 feet per side. The west edge of column 1 coincides with the coordinate x=0 feet and the north edge of row 1 coincides with y=23,000 feet. The active cells in the model grid are shown on Figure II-1 with row and column numbers labeled. The western boundary of active cells in the model corresponds to where the unconsolidated deposits meet the bedrock of the Lakeside Mountains. The eastern boundary corresponds to the approximate normal pool elevation of the Great Salt Lake. The northern and southern boundaries of the model were chosen at least 1 mile north and south of the landfill site to avoid boundary effects on the target area to be modeled. The groundwater aquifer is modeled as a single layer.



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MODEL GRID

FIGURE II-1

Boundary Conditions

The western boundary is modeled as a specified flux boundary using positive flow rate (injection) wells to simulate recharge to the unconsolidated aquifer from the bedrock and from runoff in the mountain streams of the Lakeside Mountains. The streams or drainages associated with the Lakeside Mountains are ephemeral providing runoff only during precipitation events. The eastern boundary is modeled as a specified (fixed) head boundary simulating the influence of the Great Salt Lake on the aquifer. Under existing conditions used for calibration of the model with the lake elevation at 4,195 feet, the lake boundary is at approximately x=37,000 feet (column 74) using the model coordinates. Under projected future high lake level conditions (estimated at 4,212 feet), the lake boundary is at about x=16,000 feet (column 32). The northern and southern model boundaries are modeled as no-flow boundaries simulating the west to east flow of groundwater as indicated in Technical Publication No. 42 (Stephens, 1974) published by the U.S. Geological Survey (USGS).

Layer Elevations

Top elevations of the model were determined using topographic contours from the Badger Island NW, Craner Peak, Delle, and Poverty Point USGS 7-1/2 minute quadrangles. Borings performed by Kleinfelder in 2003 indicate that the thickness of the unconsolidated deposits beneath the landfill site is at least greater than 52 feet. Additional borings completed by Applied Geotechnical Engineering Consultants in October 2004 indicate the thickness of the unconsolidated deposits to be 140 feet in the valley area of the Lakeside Mountains west of the landfill area. The bottom elevations of the model are assumed to be 100 feet below the top elevations on the west side of the model and are assumed to transition to 400 feet below the top elevations on the east of the model. The thickness of the unconsolidated aquifer is almost certainly greater than 400 feet on the east. However, the aquifer properties were modeled using hydraulic conductivity. Therefore, water levels computed by the model will be controlled mainly by the hydraulic conductivity and the bottom elevation should not have a significant impact on model results.

Great Salt Lake Elevations (Fixed-Head Boundary)

Elevations for the Great Salt Lake were obtained from the USGS Water Resources for Utah website (ut.water.usgs.gov). Near the end of 2003 when groundwater elevations below the landfill site were obtained, the elevation of the Great Salt Lake was about 4,195 feet. The historical high level of the Great Salt Lake of about 4,212 feet occurred twice in the historical record. The first time was between 1870 and 1875 and the second time was after the high precipitation years of 1980 to 1983. Based on this information, the maximum Great Salt Lake level is assumed to be 4,212 feet.

Evapotranspiration

Because of the arid conditions on the west side of the Great Salt Lake, a significant amount of groundwater is removed through evapotranspiration. Based on the presence of mud flats and other surface features, it was assumed that evapotranspiration occurs throughout the model east of the landfill site. The rate of evapotranspiration was estimated to be about 12 inches/year with a maximum evapotranspiration depth of about 5 feet below ground surface. The rate of evapotranspiration was obtained from data generated in EPA's HELP model which uses local temperature and solar radiation type climatological data, vegetative cover and soil types in generating the rate of evapotranspiration.

Recharge Estimates

The principal source of groundwater recharge to the unconsolidated aquifer was assumed to be the Lakeside Mountains to the west in the form of infiltration from runoff in mountain streams and movement of groundwater from the bedrock into the unconsolidated aquifer. Stephens (1974) indicated in Technical Publication No. 42 (TP-42) that the average percent of precipitation contributing to groundwater recharge for the periphery of the Northern Great Salt Lake Desert, which include the Lakeside Mountains, is 3%. Specific recharge was not addressed for the Lakeside Mountains in TP-42 and a recharge rate of 5% of precipitation for this area was assumed in the model to be conservative. Copies of the relevant portions of TP-42 are included in the model calculations in Appendix C.

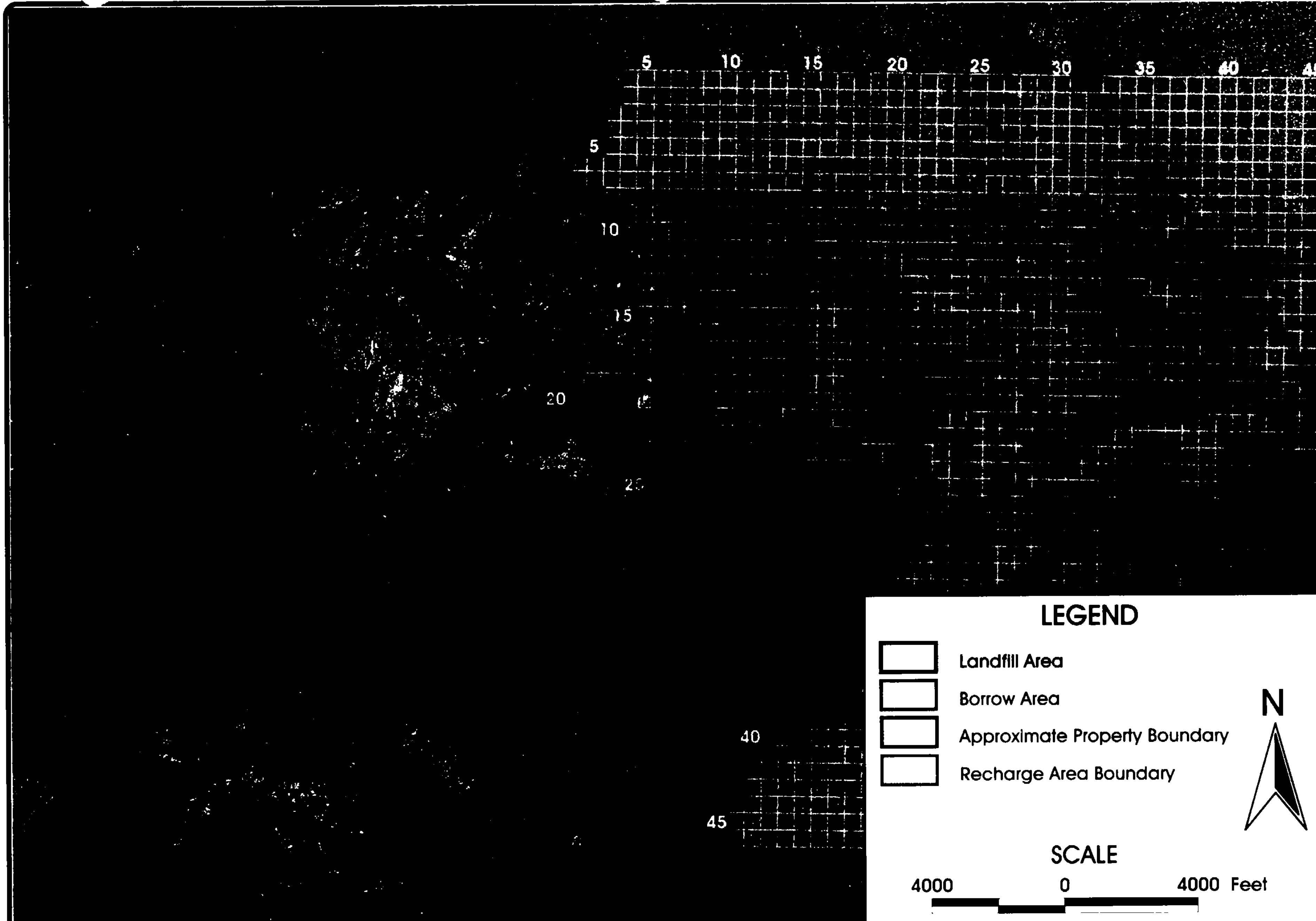
Precipitation data were obtained from the Western Regional Climate Center website maintained by the Desert Research Institute (www.wrcc.dri.edu). Using the four closest precipitation stations, the annual precipitation from 2000 to 2003 was about 6.7 inches and the annual precipitation from 1980 to 1983 was about 15.9 inches. Table II-1 summarizes the precipitation data for these two time periods.

TABLE II-1
PRECIPITATION DATA SUMMARY

Year	Annual Precipitation (inches) by Station				Estimated Precipitation (inches)
	Callister Ranch	Grantsville	Knolls 10 NE	Utah Test Range	
1980	15.73	12.67	X	X	
1981	13.07	13.06	X	X	
1982	16.55	18.45	X	X	
1983	16.50	20.78	X	X	
Average	15.5	16.2	X	X	**15.9**
2000	X	11.85	3.78	**	
2001	X	**	**	6.09	
2002	X	7.08	**	6.96	
2003	X	6.92	5.0	8.24	
Average	X	8.6	4.4	7.1	**6.7**

X Station period of record does not include this year
** Data was missing for 1 or more months during this year

Recharge from the mountains was divided into three recharge areas as shown on Figure II-2. The North Recharge Area consists of the Carter Canyon Drainage. The Central Recharge Area consists of the eastern drainages of the Lakeside Mountains south of Carter Canyon and north of Dead



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RECHARGE AREAS

FIGURE II-2

Cow Point. The South Recharge Area includes the drainage area of the Lakeside Mountains south of Dead Cow Point to the limits of the study area.

Five percent of the precipitation was multiplied by the area of each recharge area to determine the total recharge volume to the study area. This resulted in a total recharge volume of 163 acre-feet/year for calibration (2000 to 2003 precipitation data) and a total recharge volume of 385 acre-feet/year for estimation of maximum groundwater levels (1980 to 1983 precipitation data). This recharge was inserted in the form of injection wells across the west side of the model with the distribution of recharge rates based on location of canyon mouths and the recharge area tributary to the canyon mouths.

Hydraulic Conductivity and Model Calibration

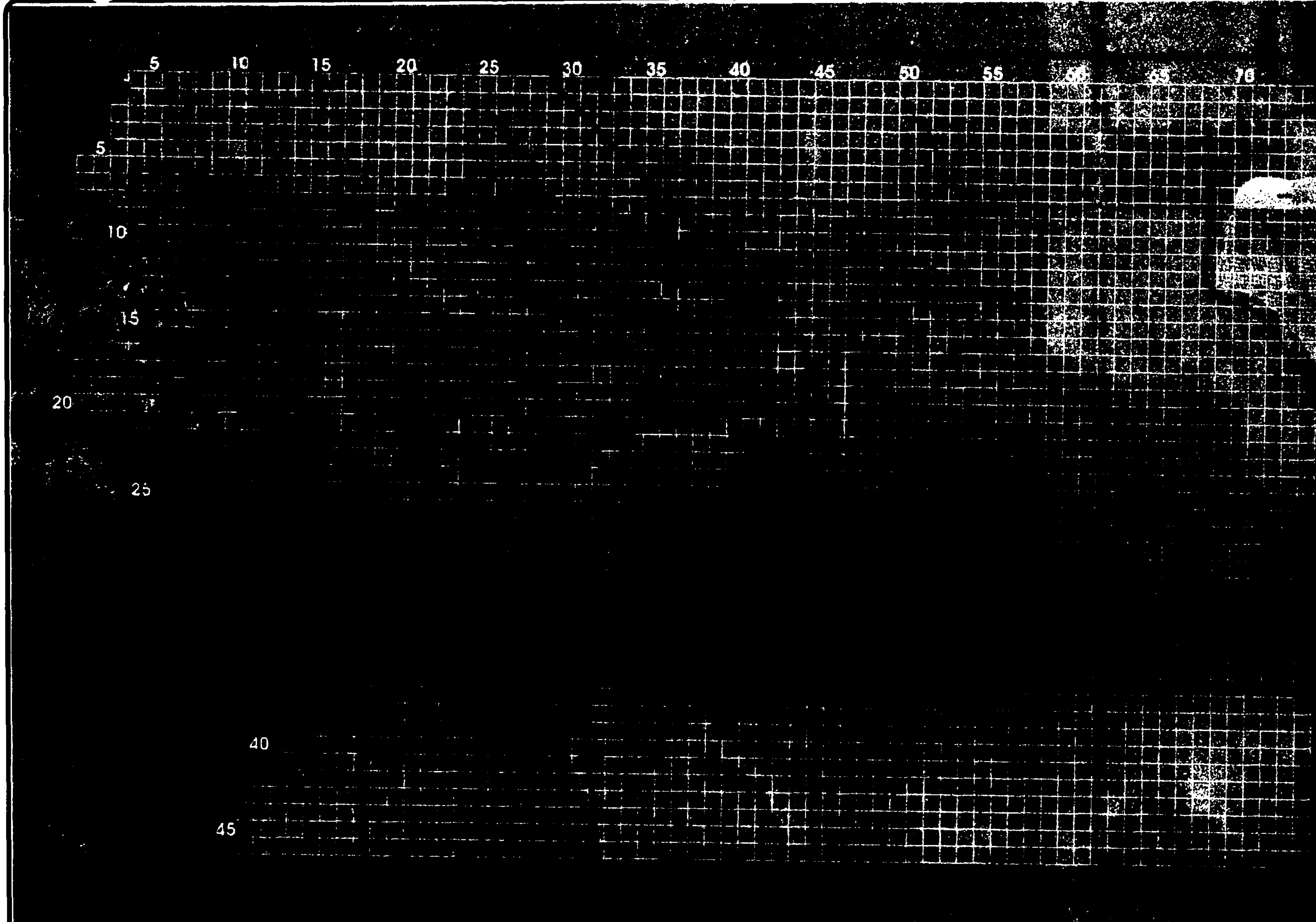
The hydraulic conductivity was assumed to vary in the model by location based on influences from mountain drainages, mud flats, or the Great Salt Lake. An initial hydraulic conductivity was assumed based based on typical values for the soil types provided in the Kleinfelder geotechnical report. The soils consist primarily of sands, silts and clays with some gravels mixed with silts and sands. "Hydrology - Water Quantity and Quality Control" presents a typical range of hydraulic conductivity values for sands, silts and clays between 0.3 feet/day and 30 feet/day. During calibration, an initial value of 7 feet/day was assumed (which is on the low side of the middle of the range of values) and the hydraulic conductivity in each zone was adjusted until the computed groundwater levels in the model approximately matched the measured groundwater levels from the 2003 Kleinfelder borehole data. Precipitation data from 2000 to 2003 and Great Salt Lake elevation data from 2003 were used during calibration. The hydraulic conductivity zones and calibrated hydraulic conductivities are shown on Figure II-3.

Figure II-4 shows the calibrated groundwater levels with the locations of groundwater observations from the boreholes drilled in 2003 by Kleinfelder. Also shown on Figure II-4 are the observed groundwater levels, computed groundwater levels, and the residual between the computed and observed groundwater levels. Computed water levels were within 2 feet of the target value in seven of the eleven observation points and were within 3 feet of the observed value in all but one observation point.

Since the south half of the landfill will be constructed first, the strength of the calibration in this area is of most importance. The computed groundwater levels were less than 2 feet above the observed levels in three of the six observation points within the southern half of the facility. The computed levels were less than 2.2 feet below the observed levels at the other three southern observation points. Therefore, the computed groundwater levels in the southern half of the facility are considered to be a reasonable representation of actual groundwater elevations.

There are five observation points in the northern half of the facility. Computed groundwater elevations in two of these were below the observed levels by 1.2 and 2.6 feet. The computed groundwater levels were 1.4, 2.6, and 4.5 feet above the observed values in the other three. The computed groundwater levels in the northern half of the facility are also considered to reasonably represent actual conditions, but the calibration may not be as close as for the southern half of the facility.

Projected Maximum Groundwater Levels



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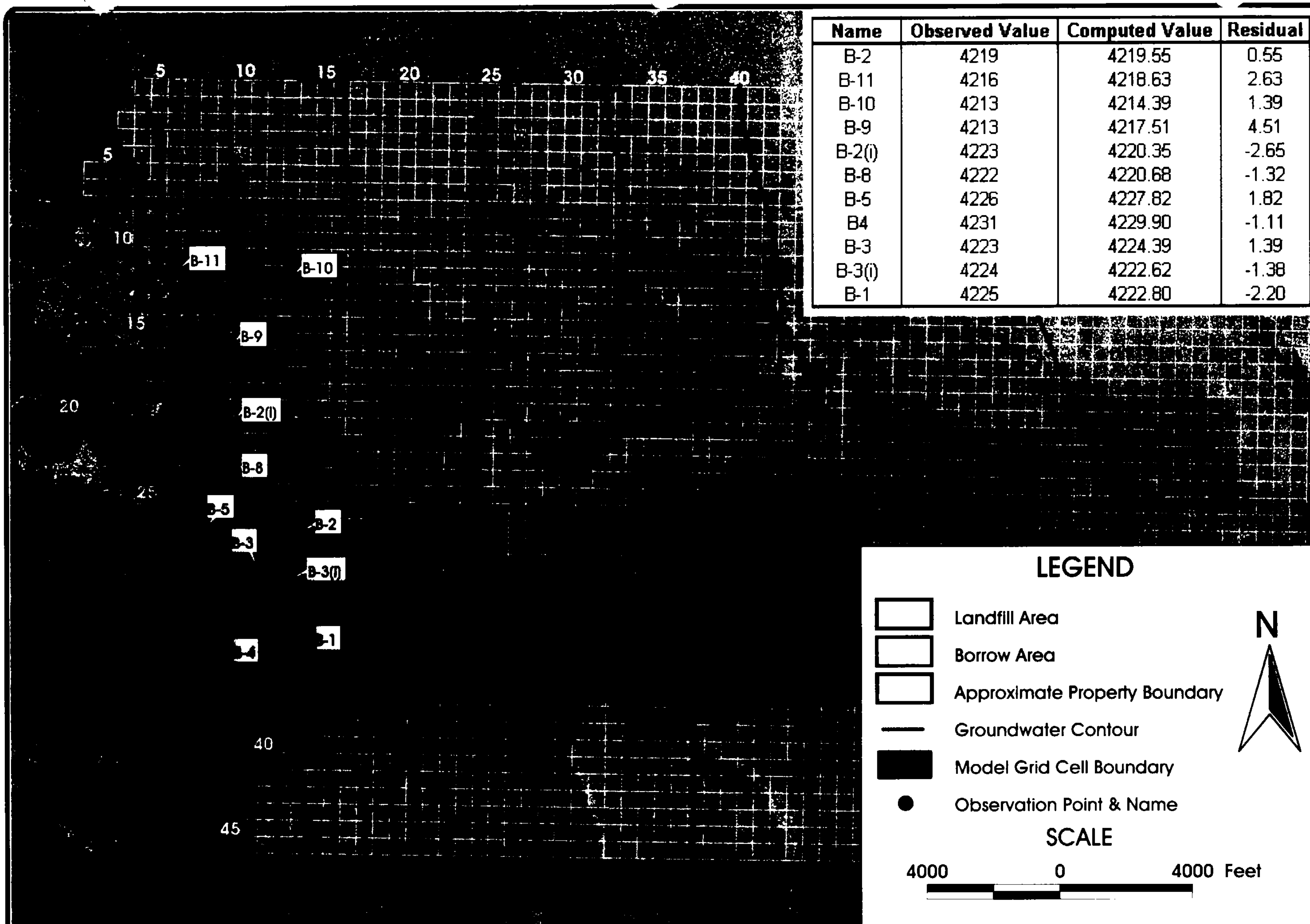
CALIBRATED HYDRAULIC CONDUCTIVITY ZONES AND VALUES

FIGURE II-3

Name	Observed Value	Computed Value	Residual
B-2	4219	4219.55	0.55
B-11	4216	4218.63	2.63
B-10	4213	4214.39	1.39
B-9	4213	4217.51	4.51
B-2(i)	4223	4220.35	-2.65
B-8	4222	4220.68	-1.32
B-5	4226	4227.82	1.82
B4	4231	4229.90	-1.11
B-3	4223	4224.39	1.39
B-3(i)	4224	4222.62	-1.38
B-1	4225	4222.80	-2.20



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CALIBRATED GROUNDWATER LEVELS WITH OBSERVATION POINTS

FIGURE II-4

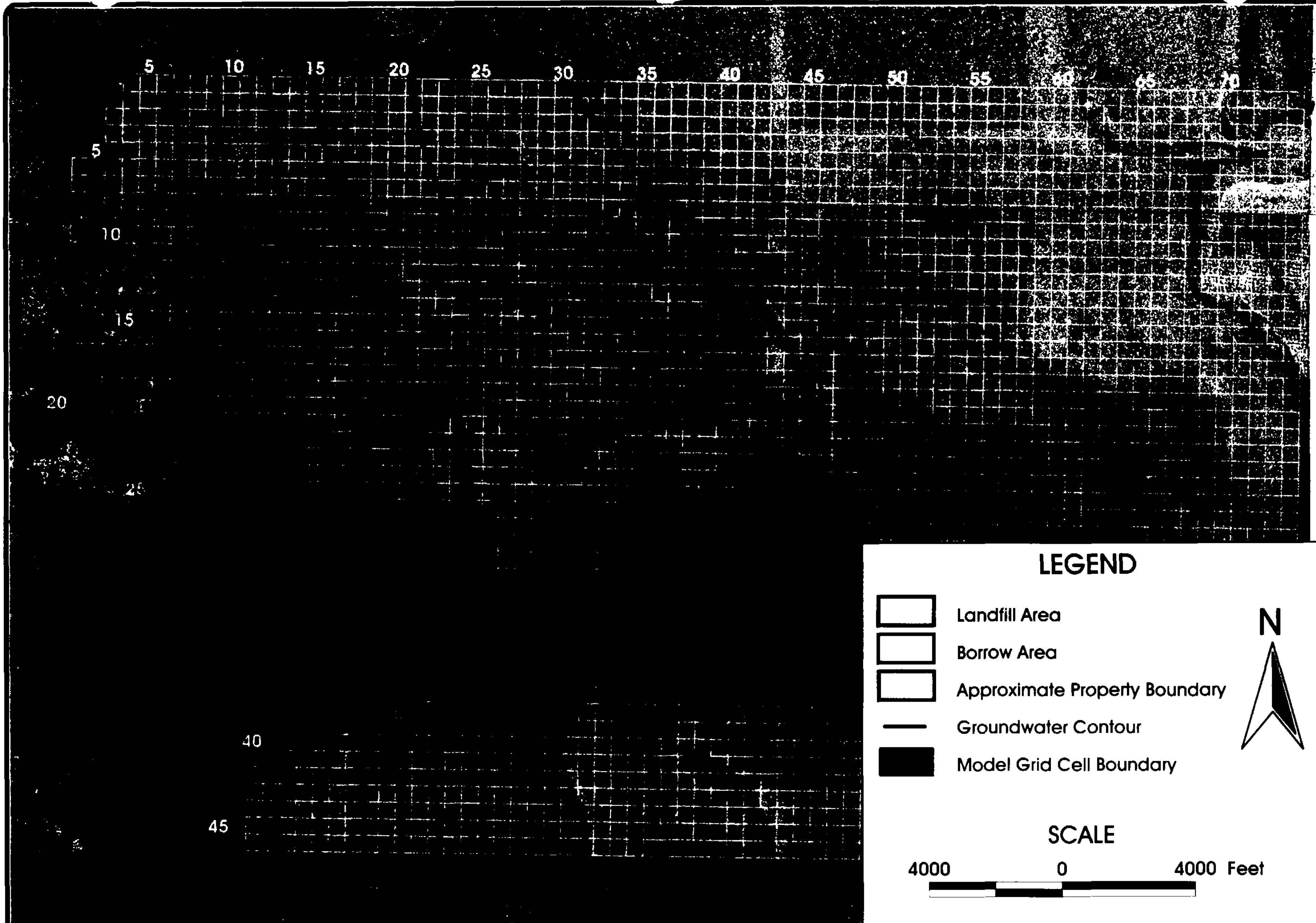
Maximum groundwater levels were computed by inserting the recharge data from 1980 to 1983 and the recorded Great Salt Lake elevation of 4212 from the year 1985 into the calibrated model and then running the model under steady state conditions. Using the highest level of the Great Salt Lake and recharge from the highest observed precipitation values in a steady state model would represent the historical worst case scenario for the landfill area. The computed maximum groundwater levels are shown on Figure II-5

Drain Trench

The computed contours shown on Figure II-5 indicate that maximum groundwater levels will be very close to the ground surface in the eastern half of the landfill site. In order to control the groundwater levels under maximum conditions, a drain trench is proposed to be constructed east of the landfills at the site. The drain trench will have a bottom width of 10 feet or more with 3H:1V (horizontal to vertical) or flatter side slopes and will have a bottom elevation of about 4,227 feet or lower. This bottom elevation was chosen to provide a minimum separation of 9.5 feet between the bottom of the landfill and the maximum groundwater level at all locations. This trench was modeled as a drain in the MODFLOW model in column 8:rows 12-16, column 9:rows 16-20, column 10:rows 20-25, column 11:rows 25-29, and column 12:rows 29-32 of the model grid. The maximum computed groundwater levels with the drain trench in place are shown on Figure II-6. The model demonstrates that construction of the drain trench will maintain lower groundwater levels even under projected maximum conditions.

Because the entire landfill would not be constructed at one time, the construction of the drain trench can be staged to coincide with landfill construction and operation. The first stage of drain trench construction may extend from the south end of the trench to the location of the drain outlet located east of the first phases of landfill construction. This location of the trench is in column 14 and rows 24 through 32 in the MODFLOW model. The computed maximum groundwater levels, with the first stage of the drain trench in place (shown on Figure II-7), demonstrate that during construction of the southern portion of the landfill, the first stage of the drain trench will maintain the lower groundwater levels used for the first phases of landfill design. The first stage of drain trench construction is expected to occur during construction and operation of the first landfill area presented in Chapter III. Construction of the drain trench will continue as construction fill materials and daily cover materials are needed.

Additional borrow materials for construction and daily cover for the entire landfill area will be obtained from the borrow area presented on the drawings to be an extension of the drain trench. This large borrow area will provide additional groundwater drainage and a larger evaporation zone for groundwater that will result in a decrease in groundwater levels below the levels projected by the MODFLOW model. Although excavation of the drain trench will occur as materials are needed for construction and operation, construction of the outlet will not be necessary until groundwater levels rise to the level of the bottom of the trench or until precipitation runoff begins to accumulate in the trench.



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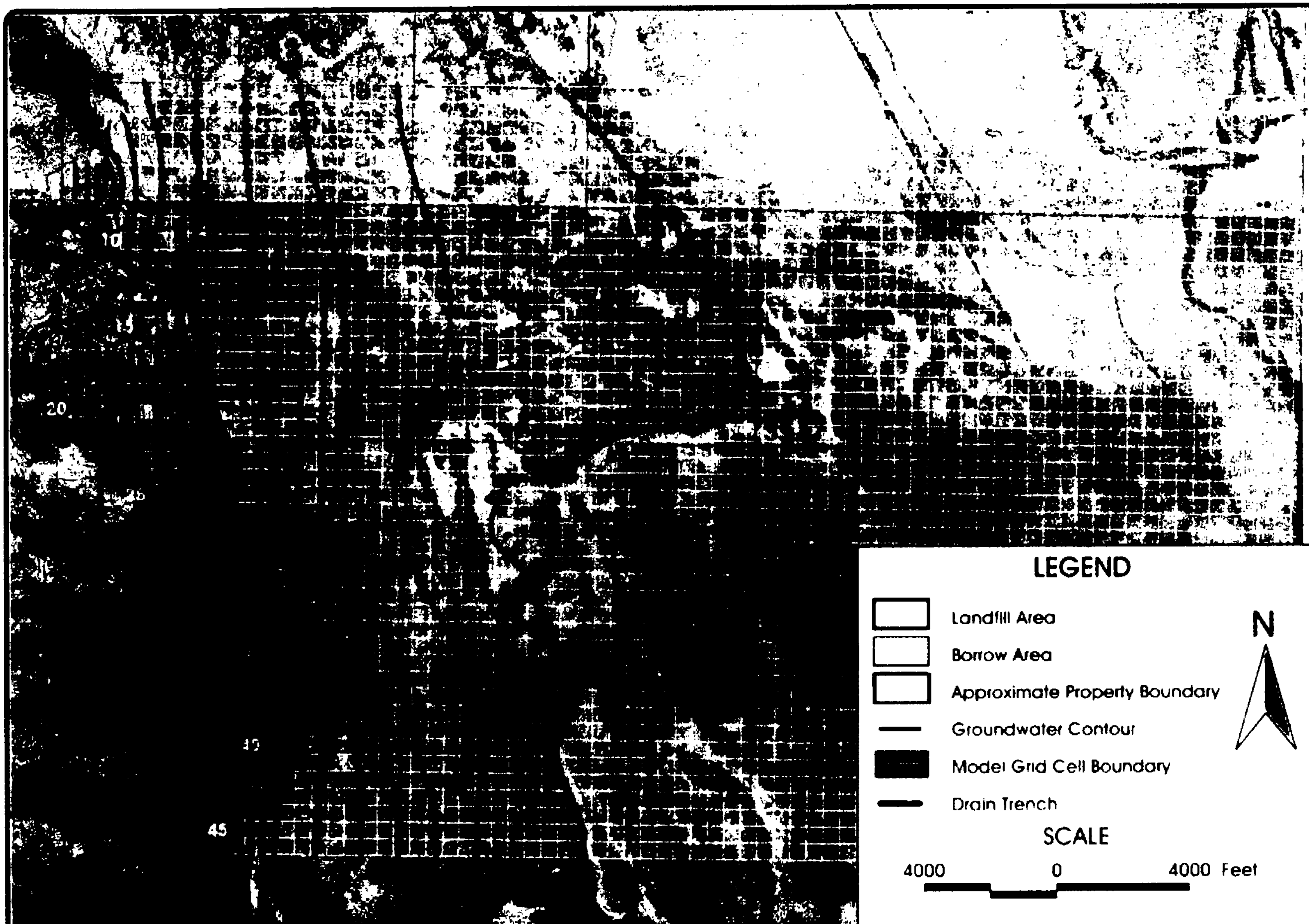
COMPUTED MAXIMUM GROUNDWATER LEVELS

FIGURE II-5



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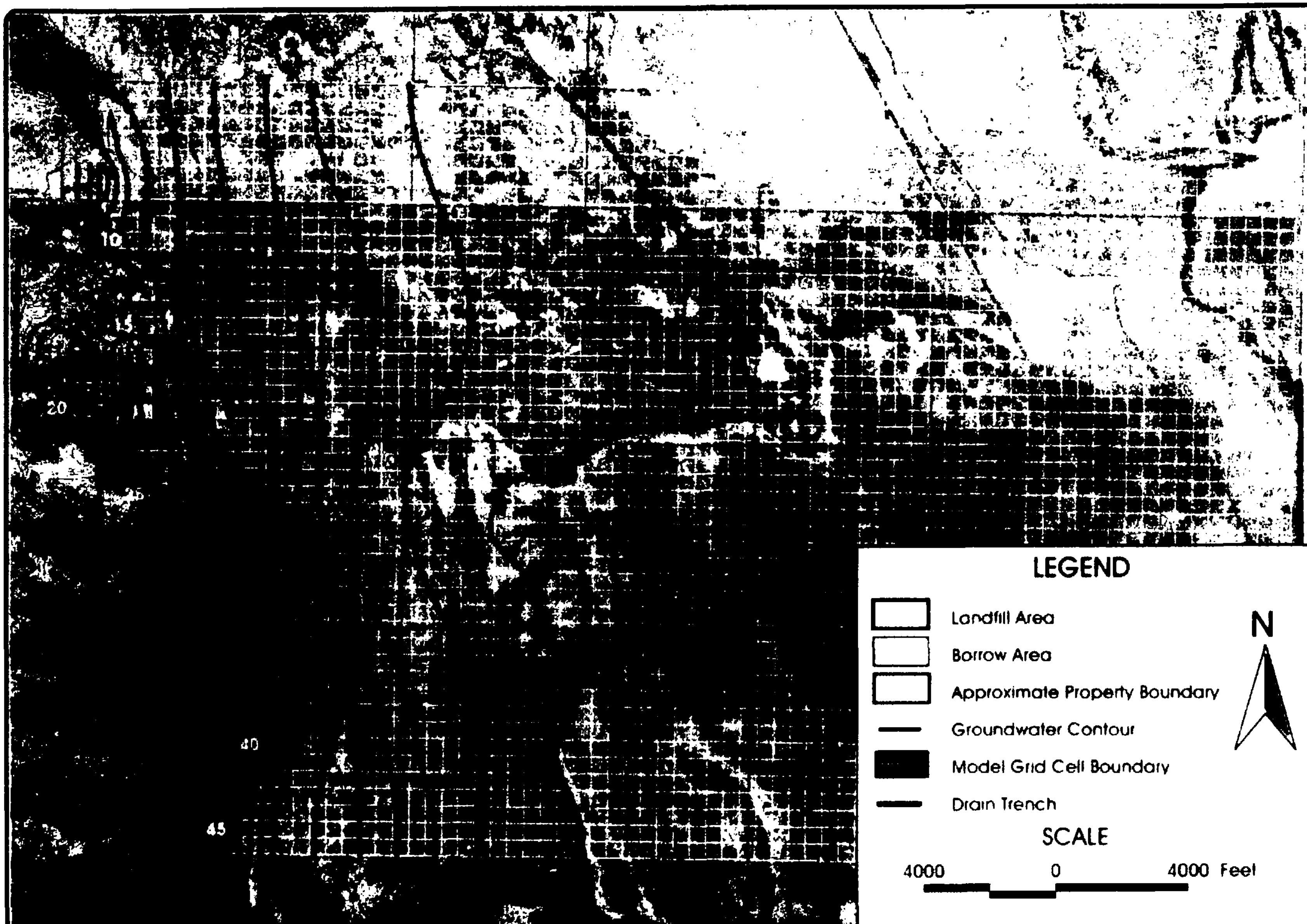
COMPUTED MAXIMUM GROUNDWATER LEVELS WITH DRAIN TRENCH

June 2005 revisi

FIGURE II-6



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COMPUTED MAXIMUM GROUNDWATER LEVELS WITH PHASE 1 DRAIN TRENCH

June 2005 Revision

FIGURE II-7

CHAPTER III

LANDFILL DESIGN

This section presents the general layout and design concept for the landfill and also presents more specific design information for the floor layout, leachate collection and removal system components and interior runoff containment. Reference should be made to the design drawings in Appendix A, geotechnical report in Appendix B, and calculations provided in Appendices D and E throughout this section.

GENERAL LAYOUT AND DESIGN

The facility consists of a landfill area formed by raised embankments along the east, north and south sides and the hill slopes along the west side of the facility. Berms are provided at a spacing of 950 feet extending from the east embankment to the west through the landfill area. These berms separate the cell into eleven individual phases or leachate management areas designated as phase 1 through phase 11 (phase 1 being the southernmost area and phase 11 the northernmost area). The sump and floor areas of each phase are designed with identical sump sizes, elevations, and floor configurations. Approximate operational areas provided by each phase are provided in Table III-1.

TABLE III-1
LANDFILL PHASE OPERATIONAL AREAS

PHASE	OPERATIONAL AREA (acres)	PHASE	OPERATIONAL AREA (acres)
1	67.2	7	93.4
2	74.1	8	92.3
3	79.6	9	130.3
4	54.6	10	41.1
5	55.7	11	44.6
6	60.2	TOTAL	793.1

The overall landfill capacity (waste mound) above the protective soil cover material placed above the lining system is about 160 million cubic yards. Assuming a daily cover quantity of 18 percent of the landfill capacity, provides for 131.2 million cubic yards of net waste capacity and a daily cover requirement of 28.8 million cubic yards. A summary of cut and fill estimated quantities are provided in Table III-2.

TABLE III-2
CUT/FILL MATERIAL QUANTITY ESTIMATES

DESCRIPTION	MATERIAL QUANTITIES (cubic yards)
Available Cut	
Cell Area	
From Construction	20.1
Clearing & Grubbing	0.7
Net Usable Cut From Cell Area	19.4
Borrow	
Total Cut	18.7
Clearing & Grubbing	0.5
Net Usable Cut From Borrow	18.2
Total Available Cut	***37.6***
Required Fill	
Embankment and Subgrade Construction	4.3
For Protective Soil Cover	2.7
Daily Cover	28.8
For Closure	3.0
Total Required Fill	***38.8***
Net Cut/Fill Balance (additional cut needed, potential import)	**1.2**

Design of the landfill area also allows for phased construction within each of the designated leachate management phases to meet ongoing capacity demands for the facility and to minimize capital expenditures based on cell capacity needs. It is anticipated that the first construction sub-phase will be approximately 20 acres (with approximately one million cubic yards of capacity) and will occur in the extreme southeast corner of the landfill area (east end of Phase 1). Subsequent construction sub-phases will extend toward the west as extensions of existing leachate management phases or toward the north into additional leachate management phases. The first sub-phase of construction for each leachate management phase will occur at the eastern end of the phase (at the sump location) to provide a system for leachate collection and removal. Details showing the concept of how construction sub-phases may end and how the tie-in for subsequent sub-phases may occur are presented in the drawings. These details present the concept only and it is expected that construction sub-phases and subsequent tie-in's will vary as ideas for tie-in's change. The important components for ending construction sub-phases are to provide for runoff containment and a continuous liner and leachate collection system.

FLOOR ELEVATION AND SLOPES

Projected future groundwater elevations presented in Chapter II and estimated settlement values presented in the geotechnical report previously submitted by Kleinfelder provided the basis for setting the lowest points (sumps) for the leachate management phases. Projected future groundwater elevations using a drain trench were used for design purposes. Estimated settlement values were also used to estimate differential settlement that may occur along the floor in establishing design slopes. Settlement projections from deeper borings provided in the Geotechnical Investigation by Applied Geotechnical Engineering Consultants (AGEC), included in Appendix B, are less than those provided by Kleinfelder. Settlement projections provided by AGEC were received after the cell design was nearly complete. Therefore, the projections provided by Kleinfelder were used for setting floor elevations and slopes resulting in a more conservative design.

The low point for each leachate management phase was established to provide a minimum separation between the liner system and the modeled projected future ground water surface of 5 feet after accounting for potential settlement. Kleinfelder projected the future settlement to be 2% to 3% of the fill height above the existing ground surface in the eastern portions of the facility and 1% to 2% of the fill height above the existing ground surface in the western portions of the facility. There will be an estimated fill height of about 20 feet to 30 feet above the existing groung surface at the location of the low point (or sump area) for each phase. Therefore, the projected settlement at these locations is 1 foot or less. A minimum separation of 9.9 feet between the liner system and the projected groundwater surface has been provided to account for settlement, and the margin of accuracy in the ground water model.

Minimum slopes used for design after accounting for potential differential settlement are: 1) Two percent minimum for the planar floor surfaces; and 2) One percent along leachate conveyance pipes. Differential settlement was estimated by determining the projected settlement resulting from an increase in fill height progressing up gradient along the width of the planar floor surfaces and up gradient along the leachate conveyance pipes. Slopes were then increased to account for the calculated potential differential settlement. The resulting design slopes are:

1) 2.75 percent for planar floor surfaces sloping downward toward the leachate collection pipes.

2) 1.0 percent for leachate conveyance pipes along the toe of inside 2H:1V slope of the east embankment sloping downward toward the sumps. These pipes parallel the contours of the fill such that there negligible change in fill height along the length of the pipes.

3) 1.7 percent downward toward the sumps for leachate conveyance pipes located below the 4H:1V closure cap slopes and extending to the west along the valleys created by the planar floor surfaces.

4) 1.2 percent downward toward the sumps for leachate conveyance pipes located below the 5 percent closure cap slopes and extending to the west along the valleys created by the planar floor surfaces

EMBANKMENTS

The east embankment has a constant top elevation of 4265 which is approximately 15 feet to 20 feet above the existing ground surface. The north and south embankments join with the east embankment at the northeast and southeast corners of the landfill area and extend west toward the west mountain area (Lakeside Mountains). An upward gradient of 1.3 percent was provided for the north embankment and upward gradients of 1.5 percent and 5 percent were provided for the south embankment (changing slope about half way along the embankment) toward the Lakeside Mountains. A top width of 25 feet has been provided for the raised embankments with 2H:1V interior slopes and 3H:1V exterior slopes.

The western boundary of the landfill area is formed by the eastern slopes of the Lakeside Mountains. Embankment fill material will be placed on the existing mountain slopes to provide an appropriate subgrade surface for placement of the lining materials. A horizontal width of about 25 feet will be provided at the top surface of the embankment fill to provide the needed width for construction (including placement of the synthetic lining materials), access around the west side of the landfill during operation, and for storm water management of precipitation run-on from the eastern slopes of the mountains and runoff from the west slopes fo the closure cap. A 2H:1V slope will be provided for the west inside slope along the western boundary of the landfill area.

LINING SYSTEM

A composite liner system is proposed for the landfill cell disposal area consisting of a Geosynthetic Clay Liner (GCL) overlain by a 60-mil HDPE goemembrane liner. The GCL is proposed in place of two feet of compacted clay liner (CCL) with a permeability no more than $1x10^{-7}$ cm/sec.

An extra GCL and 60-mil HDPE geomembrane are proposed for placement in the sump areas directly above the GCL and HDPE geomembrane placed across the rest of the cell area. This extra GCL and goemembrane provides added protection against leakage in the sump areas. Geosynthetic materials placed on the interiorslopes of the cell will consist of needle punch (or equivalently reinforced) GCL and textured geomembrane. Geosynthetic materials placed across the cell floor may be non-reinforced GCL's and smooth goemembrane.

Geosynthetic Clay Liner (GCL)

Hydraulic equivalency calculations were completed to provide a comparison between the performance of a GCL compared to two feet of a compacted clay liner. Permeability testing for the GCL materials was also completed using ground water obtained from a piezometer at the site and using permeant generated from leaching water through soils obtained from various locations of the site.

GCL Hydraulic Equivalency. Equivalency calculations were completed using comparisons between the permeability values and bentonite thickness data for the GCL as compared to two feet of CCL with a permeability of 1×10^{-7} cm/sec. Procedures used for this evaluation are based on a technical paper published by R.M. Koerner entitled "Technical Equivalency Assessment of GCL's to CCL's." Table III-3 provides a comparative tabulation of required permeability and hydrated thickness values required for the GCL materials to show equivalency with two feet of CCL at a permeability of 1×10^{-7} cm/sec. GCL materials used for construction should be tested and certified to demonstrate a combination of thickness and permeability characteristics presented in the table.

An equivalency evaluation was also made using the Hydrologic Evaluation of Landfill Performance (HELP) computermodel developed by the U.S. Environmental Protection Agency. Results from the HELP model show a leakage rate through the bottom lining system of 0.375 cubic feet per year using CCL material meeting minimum regulatory requirements and 0.169 cubic feet per year using a GCL of equivalent hydraulic characteristics to the CCL material.

TABLE III-3
COMPARATIVE VALUES FOR GCL'S FOR
HYDRAULIC EQUIVALENCY WITH CCL'S

Permeability	Thickness		
(cm/sec)	(mm)	(cm)	(inches)
1.9×10^{-9}	4.0	0.40	0.157
2.4×10^{-9}	5.0	0.50	0.197
2.9×10^{-9}	6.0	0.60	0.236
3.4×10^{-9}	7.0	0.70	0.276
3.8×10^{-9}	8.0	0.80	0.315
4.3×10^{-9}	9.0	0.90	0.354
4.8×10^{-9}	10.0	1.00	.0394
5.2×10^{-9}	11.0	1.10	0.433
5.7×10^{-9}	12.0	1.20	0.472
6.1×10^{-9}	13.0	1.30	0.512
6.6×10^{-9}	14.0	1.40	0.551

GCL Ground Water and Leachate Compatibility.

Compatibility tests were conducted by an independent laboratory for CETCO (a manufacturer and supplier of GCL materials) and by AGEC using ground water obtained from below the site and using leachate generated using soils obtained from the site. The compatibility tests were conducted to determine if the sodium content in the ground water and in the soils to be used for construction will reduce the integrity of the GCL.

Leachate generated from soils obtained at the site was used to conduct a 30-day permeability test by the independent laboratory for CETCO. The test results show a permeability of about 5×10^{-10} cm/sec.

Tests were also conducted by AGEC to determine the compatibility of GCL materials with the groundwater at the site and with soils that will potentially be used for construction. Atterberg limits were first obtained to determine the plasticity of the bentonite material obtained from GCL samples of two suppliers. Atterberg limits were determined using distilled water, a sample of groundwater obtained from a piezometer at the site, and from leachate water obtained from four soil samples at the site. A permeability test was then conducted on the GCL material that appeared to be impacted the most by the groundwater and water leachates and using leachate from the soil sample showing the greatest impact on the GCL material. This was done to obtain worst case results from the available material and water samples. Leachate from AGEC's soil sample A had the greatest impact on the Atterberg limits. A permeability of 1.5×10^{-9} cm/sec. was obtained from the permeability test conducted which is a better value than the values listed in the table. This is also a lower value than the GCL permeability specification of 5×10^{-9} cm/sec published by the two suppliers. Test results are provided in the Geotechnical Investigation report included as Appendix B.

HDPE Geomembrane Liner

HDPE geomembrane is proposed for use as the synthetic liner system above the geosynthetic clay liner. The floor area will consist of 60-mil smooth HDPE geomembrane and the interior slopes and phase division berms inside the landfill area will consist of 60-mil textured HDPE geomembrane to increase slope stability for materials placed on the side slopes above the HDPE geomembrane.

LEACHATE COLLECTION AND REMOVAL SYSTEM (LCRS)

A leachate collection and removal system (LCRS) will be constructed consisting of geonet placed directly over the HDPE geomembrane liner system overlain by non-woven geotextile filter fabric. Perforated leachate conveyance pipes will be placed in the valley areas formed by the planar surfaces of the floor area. These leachate conveyance pipes will collect and convey leachate from the cell floor to the sumps for removal. EPA's computer HELP model was used to obtain leachate quantities for design of the LCRS.

HELP Model

EPA's Hydrologic Evaluation of Landfill Performance (HELP) model is a quasi-two-dimensional hydrologic computer model used for conducting water balance analyses of landfills, cover systems and other solid waste containment systems. The model accepts weather, soil and design data, and uses solution techniques that account for the effects of surface storage, snowmelt, runoff, infiltration, evapotranspiration, vegetative growth, soil moisture storage, lateral subsurface drainage, leachate recirculation, unsaturated vertical drainage, and leakage through soil, geomembrane and/or composite liners.

Climatologic data (precipitation, evaporation, solar radiation, and temperatures) for the modeling effort were obtained from default data contained within the HELP model software corresponding to the Salt Lake area. Climate data used were compared with average temperature and precipitation data reported for Dugway and the Saltair Salt Plant in the Western Regional Climate Center database. In general, the comparison of data showed the model generated data to be slightly conservative, but compared closely with data from Dugway and the Saltair Salt Plant. This result is a conservative, but reasonable, projection of leachate rates for design of the LCRS.

Six layers were defined in the help model corresponding to municipal waste material, soil cover, non-woven geotextile, geonet, HDPE geomembrane and GCL to represent the open cell area. An additional three layers were added above the waste consisting of HDPE geomembrane, soil cover material, and the erosion protective layer to represent closed portions of the landfill. Model default data were used to define the physical properties of the individual design layers. Leachate quantities were generated forthe landfill assuming no waste, and waste thicknesses of 10 feet, 50 feet, 100 feet, and 200 feet to simulate various stages of landfill operation. Table III-4 provides the leachate quantity values generated by the HELP model that were used for LCRS design.

TABLE III-4
HELP MODEL GENERATED LEACHATE RATES

Waste Height (feet)	Peak Daily Leachate		Annual Average Leachate	
	(inch)	(gal./acre)	(inches)	(gal./acre)
No Waste	0.139	3,774	1.613	43,797
10	0.215	5,838	2.702	73,366
50	0.209	5,675	2.702	73,366
100	0.242	6,571	2.702	73,366
200	0.222	6,028	2.702	73,366

Geonet

Geonet will be placed on the planar surfaces of the cell floor to collect and convey leachate from the floor area to leachate conveyance pipes that convey the leachate to the sumps for removal. The peak daily leachate rate of 0.242 inches was used to determine the required geonet capacity. Designing the geonet assuming a one-foot wide section of geonet extending from the leachate conveyance pipe to the upper end of the widest planar surface will provide the longest flow path and a typical design that can be applied to all areas of the floor.

The longest flow path in the geonet is between 130 and 140 linear feet which is the floor surface adjacent to the leachate conveyance pipe extending west of the center of the sumps. Using the 140 feet of flow path length and a one-foot width gives a leachate area of 140 square feet. Applying the leachate rate of 0.242 inch to the leachate area gives a project leachate flow through the geonet of 2.82 ft^3/ft-day.

Designing with Geosynthetics, by Robert Koerner, suggests several safety factors that will be applied the the leachate rate to obtain a design capacity for the geonet. These safety factors include: 1) a safety factor for intrusion of adjacent geosynthetics into the geonet (SF_{in}=1.5); 2) a safety for creep deformation of the geonet (SF_{cr}=1.5); and 3) a a safety factor for biological and chemical clogging (SF_{bcc}=2.0). Koerner also recommends a safety factor for the design-by-function concept (SF_{n}=1.5) to be included as an additional safety factor to obtain a resulting safety factor (SF_{res}= 1.5 x 1.5 x 2.0 x 1.5 = 6.75) to be used for design of the geonet. Applying this resulting safety factor to the leachate rate gives a design leachate rate of 19.03 ft^3/ft-day. A required geonet transmissivity of 1.023 x 10^{-3} m^2/sec was obtained using the design leachate rate.

The overburden loading, hydraulic gradient, and the boundary conditions for the geonet have a large influence on the transmissivity. Estimated overburden loadings vary from about 2,500 pounds per square foot (psf) above the sump to about 10,000 psf at the breakline of the closure cap from the 4H:1V slopes to the 5% slope, to about 20,000 psf along portions of the west side of the closure cap. There is a variety of manufacturers, thickness, and types of geonets with different structural and transmissivity characteristics. Geonets installed as part of the LCRS should be tested prior to installation and laboratory results should be provided by manufactures to demonstrate that transmissivity values are equal to or greater than 1.023 x 10^{-3} m^2/sec at the estimated loading, boundary, and hydraulic gradient conditions for each construction phase of the landfill.

Geotextile Filter Fabric

Criterial published in the "Geotextile Engineering Manual" by the U.S. Department of Transportation and in "Designing with Geosynthetics" by Robert M. Koerner were used to determine geotextile filter fabric design for filtering on-site soils from the LCRS. Gradation properties used for the calculationswere obtained from Klienfilder's geotechnical report of the site. A filter material consisting of non-woven geotextile filter fabric will be placed above the LCRS and around the leachate conveyance piping on the cell floor to provide a filter layer between the soil cover material and the LCRS. Physical properties required for the geotextiles are summarized in Table III-5. Physical properties provided in Table III-5 are available typically with 8 oz. and 10 oz. non-woven geotextiles.

TABLE III-5
REQUIRED PROPERTIES FOR GEOTEXTILE FILTER FABRIC

Property	Standard
Equivalent Opening	≤0.2 mm (#80 Sieve)
Permeability	≥10-2 cm/sec
Grab Tensile Strength	≥200 lbs. (up to 200 feet of waste pile, 16,700 pcf) ≥246 lbs. (up to 250 feet of waste pile, 20,000 pcf)
Burst Strength	≥350 psi

Leachate Conveyance Pipes

Leachate conveyance pipes are designed along the valleys of the cell floor that are formed by the intersection of the planar surfaces on the floor. These leachate collection pipes receive leachate from the geonet component of the leachate collection system and convey the leachate to the sumps for removal.

A maximum leachate rate to the pipes was determined using the maximum width offloor area where leachate will be collected in the geonet and conveyed to the pipes. The maximum width is 280 feet consisting of 140 feet to the north and 140 to the south of the center pipe which extends to the west from the center of each sump. Using the design leachate rate of 0.242 inch/day applied over an area of 280 ft^2 gives a rate of leachate entering the conveyance pipes of 0.029 gpm per foot of pipe length.

Eighty percent of the maximum flow capacity was assumed for the actual capacity of the pipes calculated using Manning's equation and a Manning n roughness value of 0.016. Flow capacity in an 8-inch diameter pipe is 127 gpm which is sufficient capacity to receive leachate for up to 4,400 feet of pipe length. Flow capacity in a 6-inch diameter pipe is 59 gpm which is sufficient capacity to receive leachate for up to 2,000 feet of pipe length. Therefore, for each cell phase or leachate management area, 6-inch diameter or larger perforated pipe can be used for the western most 2,000 feet of pipe length. None of the cell phases has a length greater than 4,400 feet, therefore, 8-inch diameter or larger perforated pipe may be used to extend from the sumps to the east end of the 6-inch diameter (or larger) pipes.

Landfill Leachate Withdrawal Pipes

Leachate withdrawal pipes were evaluated for wall crushing, wall buckling, and ring deflection using procedures published in "Design and Engineering Guide for Polyethylene Piping" by Rinker Materials and "Plexco/Spirolite Engineering Manual 2. System Design", by Chevron Chemical Co. Overburden loadings were determined based on the loading over the low point (sump) of the leachate management phases of the landfill. The leachate withdrawal pipes with a Standard Dimension Ratio (SDR) of 15.5 provide sufficient strength to resist wall crushing, wall buckling, and will not experience excessive ring deflection.

Leachate Ponds

Leachate will generally be contained and managed within the landfill and pumped from closed phases or phases nearing closure to phases where capacity is provided for containment of leachate. When the distance is too great for leachate to be moved from closed phases to open phases of the landfill, double lined leachate ponds will be constructed where leachate can be contained and evaporated or stored for re-circulation, compaction, or dust control in the landfill.

The proposed leachate pond has top dimensions of 100 feet square, 3H:1V sideslopes and is approximately 10 feet deep. This provides a storage capacity of 351,300 gallons (1.08 acre-feet) with one-foot of freeboard and a total capacity of 433,800 gallons (1.33 acre-feet) to the top. Results from the HELP model predict a peak day leachate volume from a closed cell of 225 gallons per acre. Based on predicted peak-day leachate volumes generated by the HELP model for a closed cell, each pond has capacity to contain leachate from 1,560 acres and maintain one-foot of freeboard.

Leachate pond lining systems will include a composite secondary (bottom) lining system constructed of GCL overlain by a 60-mil HDPE geomembrane. A leak detection and removal system consisting of a geonet, a sump, and a leachate withdrawal pipe will be placed above the secondary lining system. A primary (upper) lining system consisting of 60-mil HDPE geomembrane will be placed above the leak detection system above which the leachate will be stored.

RUNOFF CONTAINMENT

Precipitation runoff from the waste material in open areas of the landfill will be contained and managed within the landfill. Containment areas will be formed on waste surfaces and/or by maintaining waste set-back areas where runoff water will be contained between phase berms and the waste material. Sufficient capacity will be maintained in these areas to contain runoff from the 25-year 24-hour precipitation event as required by the regulations.

The required containment capacity is determined by obtaining a precipitation runoff depth using the SCS curve number methodology and applying that runoff depth to the open area of the landfill. A 25-year 24-hour precipitation depth of 2.06 inches was obtained from NOAA Atlas 14. A curve number of 82 was selected to represent conditions within the landfill representative of the daily soil cover material using on-site soils. On site soils are within the hydrologic soil group "type B" soils. Surface conditions were assumed to represent that of a dirt road (including right-of-way) provided in table 2-2a of U.S. Department of Agriculture Technical Release 55. A curve number of 82 should be representative, but slightly conservative, since daily cover materials are typically placed with dozers and landfill compactors that provide individual depressions across the surface that increases interception storage.

Calculations show a required runoff containment capacity of 0.06 acre foot (2,613 cf) per acre of open cell area. Therefore, for the first phase of construction the containment capacity for approximately 20 acres is 1.2 acre-feet (52,272 cf). This containment capacity may be provided in a number of ways including:

1. Maintaining a waste set-back from the inside slope of the cell.
2. Creating a pond area on the waste surface.
3. Maintaining ditches between the waste and the interior slope of the cells.
4. Providing separate lined runoff containment storage areas outside the landfill operating area.
5. A combination of the above or any other method that will provide the required containment capacity.

We recommend that facility operators provide a minimum freeboard of two feet within the containment areas. Runoff water collected in the containment areas may be re-circulated in the landfill by using the water for dust control and compaction.

GROUND WATER MONITORING WELLS

Monitoring wells are planned along the eastern side of the landfill area to monitor ground water quality during the operational life and closure/post closure period for the landfill. Currently, twelve monitoring wells are planned consisting of eleven monitoring wells down-gradient from each of the eleven sumps and one monitoring well in the valley area up-gradient from phase 1. The monitoring well up-gradient from the phase 1 area and the monitoring wells down-gradient from phases 1 and 2 have been installed.

Monitoring well locations were selected to provide approximately 950 feet of spacing between the wells to allow for ground water monitoring within 475 feet of any point along a line parallel to the cell embankment and liner system. The monitoring wells are also located approximately 75 feet away from the bottom exterior toe of the cell embankment to allow for construction, maintenance, and other equipment to access the embankment and slopes without risking potential damage to the monitoring wells.

GEOTECHNICAL INVESTIGATION

Applied Geotechnical Engineering Consultants (AGEC) completed a geotechnical investigation for the specific design. The complete geotechnical investigation report is provided in Appendix B. Conclusions presented in the report indicate:

1. The natural soil and bedrock at the site are suitable for support of the proposed landfill disposal facility.
2. Exterior slopes of 3H:1V and interior cut and fill slopes of 2H:1V may be used for the base of the landfill facility.
3. The natural soil is suitable to use in construction of the proposed embankment.
4. A geosynthetic clay liner (GCL) will provide appropriate stability along with the other synthetic materials for the interior of the landfill.
5. Permeability tests conducted on the GCL, using worst case conditions from GCL and permeant samples obtained and generated, resulted in a permeability of $1.5x10^{-9}$ cm/sec.
6. The subsurface soil investigated under the landfill area during the study by AGEC and from information presented by Kleinfelder was found to not be susceptible to liquefaction at an acceleration with a 5% probability of exceedance within 50 years.

The conclusions presented are based on data obtained from the Kleinfelder geotechnical report and from additional soil borings and laboratory testing conducted by AGEC. The report by AGEC should be referred to for a more detailed presentation of testing conducted, material strengths, interface friction angles, and stability safety factors under static and seismic conditions.

CHAPTER IV

LANDFILL CLOSURE DESIGN

This section presents the general layout and design concept for the landfill closure system and also presents more specific information regarding stability of the closure system. Storm water management and erosion protection are presented in detail in Chapter V. Reference should be made to the design drawings in Appendix A, geotechnical report in Appendix B, and calculations provided in Appendices D and E throughout this section.

GENERAL LAYOUT AND DESIGN

The final waste mound with the overlying daily cover material provides the sub-grade to the closure cap system. A final cover system consisting of a Geosynthetic Clay Liner (GCL), 60-mil HDPE textured geomembrane and 2 feet of cover material is placed above the waste mound. The two feet of cover material includes soil fill and an erosion protective layer consisting of either six inches of top soil and vegetation or three inches of stone mulch material. A discussion of the erosion protection measures is provided in Chapter V. Although the U.S. Environmental Protection Agency approves a closure system consisting of an 18-inch thick layer of 1 x 10^{-5} cm/sec infiltration layer overlain by the flexible membrane liner (60-mil HDPE textured geomembrane for this design), Wasatch Regional is providing a GCL to comply with the Utah Division of Solid and Hazardous Waste interpretation of the closure design requirements provided in 40 CFR Part 258.

Closure slopes

Waste mounding and the overlying closure cap extends up on a 4H:1V slope from the top of the embankments around the perimeter of the landfill area. The waste mound extends up from the top inside edges of the embankments and the two feet of cover will be placed above this waste mound. Intermediate benches (25-feet wide) are designed in the 4H:1V slopes to provide for intermediate storm water collection and conveyance necessary for erosion protection on the slopes. The east side of the waste mound and closure cap provides grade control for the height of waste and closure system across the rest of the landfill area. The waste mound rises to an elevation of 4365, or 100 feet (with the closure cover at 4367 or 102 feet) above the top of the east embankment. The waste mound and closure cap then break grade to a five percent slope extending toward the west.

The north, south, and west slopes extend upward on 4H:1V slopes from the top of the embankments to intersect with the top surface as it extends west on the five percent slope. Intermediate benches are also placed in the 4H:1V slopes where slopes are of sufficient length that the intermediate benches are required for erosion protection.

Sub-Surface Drainage

Some storm water may infiltrate through the cover system and collect on the surface of the HDPE geomembrane. A drainage system consisting of two parallel perforated drain pipe with a separation of about 100 feet is provided under the storm water containment berm at the top of the east 4H:1V slope and 100 feet up-gradient from the containment berm. The drain pipes are placed in drain rock with a geotextile filter fabric wrap around the drain rock. These pipes are provided to drain free water from the soils placed on the top 5 percent slope of the cap above the HDPE geomembrane. Additional perforated drain pipes will be placed under the bench drainage ditches located on the 4H:1V perimeter slopes.

Sub-surface drain pipes located along the top east side of the 5 percent cap slope convey water collected down the top 4H:1V slope in solid pipe and discharge the water into the storm water inlet boxes located on the top bench. Sub-surface drain pipes located under the bench drainage ditches convey the water collected to solid 3-inch down drains and discharge the water collected at the exterior toe of the cell embankment.

STORM WATER MANAGEMENT

The storm water management system consists of a 5 percent slope at the top of the landfill that directs precipitation runoff from the top surface of the closure cap toward the east. Runoff water is then collected and directed to storm water down drains (or downspouts) consisting of inlet boxes and parallel 24-inch diameter pipes. The downspouts convey the stormwater from the top of the closure cap to the exterior toe of the embankment where a drainage channel, connecting storm drainage pipes, or a combination of drainage channel and storm drainage pipes will convey the runoff to the storm water basin.

Intermediate benches are located on the 4H:1V perimeter slopes of the closure cap primarily to shorten the length of the 4H:1V slopes for erosion control purposes. These intermediated benches also provide storm water conveyance ditches that convey storm water runoff collected in the ditches to inlet boxes and to 15-inch diameter downspout pipes located at low points along the benches. Storm water is then conveyed to the exterior toe of the embankment slopes and conveyed to the storm water pond in the storm drainage channels and pipes provided for drainage from the top of the closure cap.

The storm water management system associated with the closure cap is designed for the 100-year 24-hour precipitation event. Design of the storm water management system, including the hydrology, hydraulic design of the downspout pipes and erosion control associated with the closure cap is presented in detail in Chapter V.

STABILITY

The stability of the closure cap design was evaluated by AGEC based on information provided in the Kleinfelder geotechnical report and on additional soil borings and laboratory testing conducted by AGEC. The complete geotechnical investigation report is provided in Appendix B. Conclusions presented in the geotechnical report indicate that natural soils are suitable for construction of the closure cap and that the closure cap, as designed, has adequate stability safety factors under both static and seismic conditions.

The report by AGEC should be referred to for a more detailed presentation of testing conducted, material strengths, interface friction angles, and stability safety factors under static and seismic conditions.

CHAPTER V

STORM WATER MANAGEMENT

Channels will be constructed to manage storm water from the Lakeside Mountains west of the facility. Berms on the closure cap will convey storm water to downspouts that will take the water off the landfill closure cap. A hydrologic analysis was completed in order to determine peak flow rates to use for the design of the channels, downspouts and erosion control.

HYDROLOGY

Hydrologic calculations were completed for the tributary area to the landfill and the closure cap to determine peak runoff for the design. The SCS (Soil Conservation Service) curve number methodology was used in conjunction with the Army Corps of Engineers HEC-1 hydrology computer model to predict peak flows from the closure cap. The methodology for predicting peak flows requires a delineation of the sub-basins generating runoff, determination of a curve number to be used, a precipitation rate, a storm distribution, and a calculation of the time of concentration and lag time.

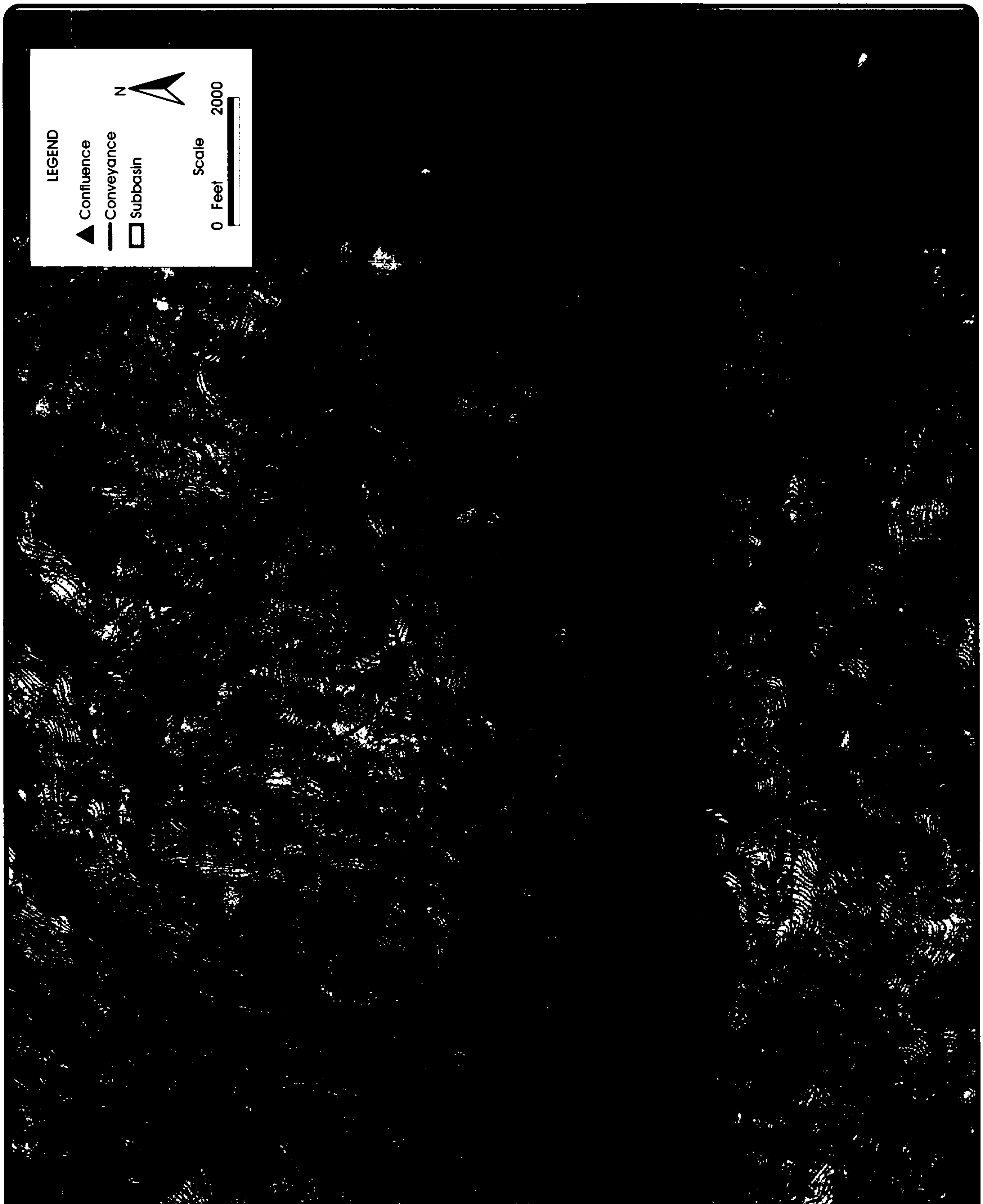
Off-Site Run-On Storm Water

Storm water that originates from outside the landfill facility will need to be diverted in order to prevent water from entering the facility or from eroding the closure cap.

Methodology. Storm drainage channels extending to the north and to the south will collect and divert storm flows from the Lakeside Mountains around the landfill facility. Tributary areas to these channels were delineated based on USGS topographical maps. The tributary areas were then divided into sub-basins as shown in Figure V-1 in order to allow for a progressive design instead of designing the entire channel for the entire flow from all combined sub-basins.

Curve numbers were determined based on the hydrologic soil type and soil vegetation cover as shown. The hydrologic soil type is a general indication of the soil's infiltration capacity. Soils are assigned a hydrologic type of A, B, C, or D by the Natural Resource Conservation Service (NRCS). Soils of hydrologic soil type A have the highest infiltration rate, and therefore produce the least amount of runoff. Soils of hydrologic soil type D have the lowest infiltration rate, and therefore produce the highest amount of runoff. Most of the soils within the tributary area are hydrologic soil type D with some type B soils. The soil vegetation cover and conditions were assumed based on information given in the NRCS study "Soil Survey of Tooele Area, Utah" and verified by a field visit on October 26, 2004. The cover conditions were combined with the hydrologic soil type to produce a curve number based on Table 2-2d of Technical Release 55. Because some sub-basins contained several different soil types and covers, an area weighted curve number was applied to each sub-basin.

The lag times (T_L), defined as the time to the hydrograph peak, were calculated by using the time of concentration (T_c) and the equation $T_L = 0.6T_c$. The time of concentration (the time it takes for runoff to travel to a point of interest from the hydraulically most distant point) was calculated using the criteria found in Worksheet 3 in TR-55 "Urban Hydrology of Small Watersheds".



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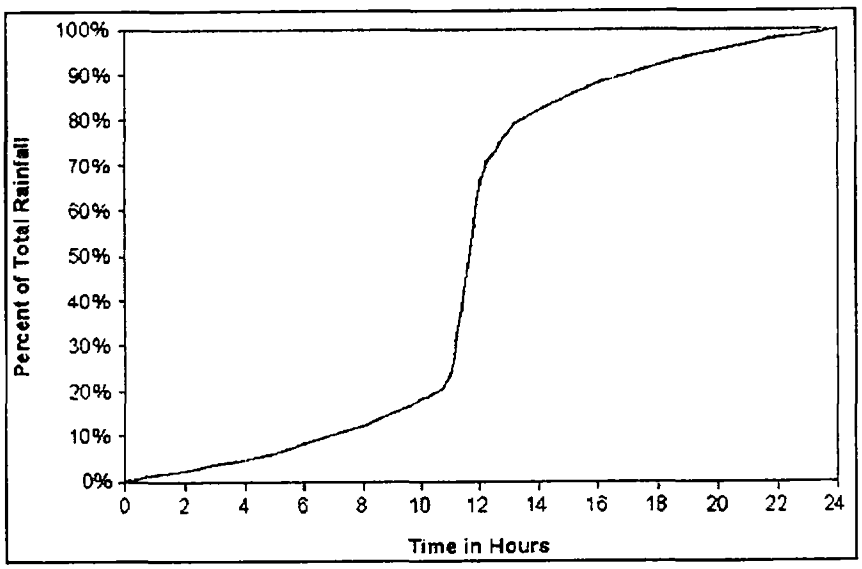
OFF-SITE HYDROLOGY MODEL

FIGURE V-1

The SCS Type II Distribution was used to model a 24-hour 100-year storm. Part 258 of the Code of Federal Regulations Title 40 Chapter 1 entitled "Criteria for Municipal Solid Waste Landfills" states that the landfill must contain "a run-on control system to prevent flow onto the active portion of the landfill during the peak discharge from a 25-year storm". Although the requirement is only a 25-year storm, a 100-year storm event was used in order to provide a more capable design that will provide better storm water management and protection of the landfill and its closure cap. The SCS Type II Distribution is shown in Figure V-2. The rainfall amount was taken for the higher elevations associated with the east slopes of the Lakeside mountains from the "Point Precipitation Frequency Estimates from NOAA Atlas 14". The value for a 100 year - 24 hour event was 2.61 inches.

FIGURE V-2
SCS TYPE II STORM DISTRIBUTION CURVE



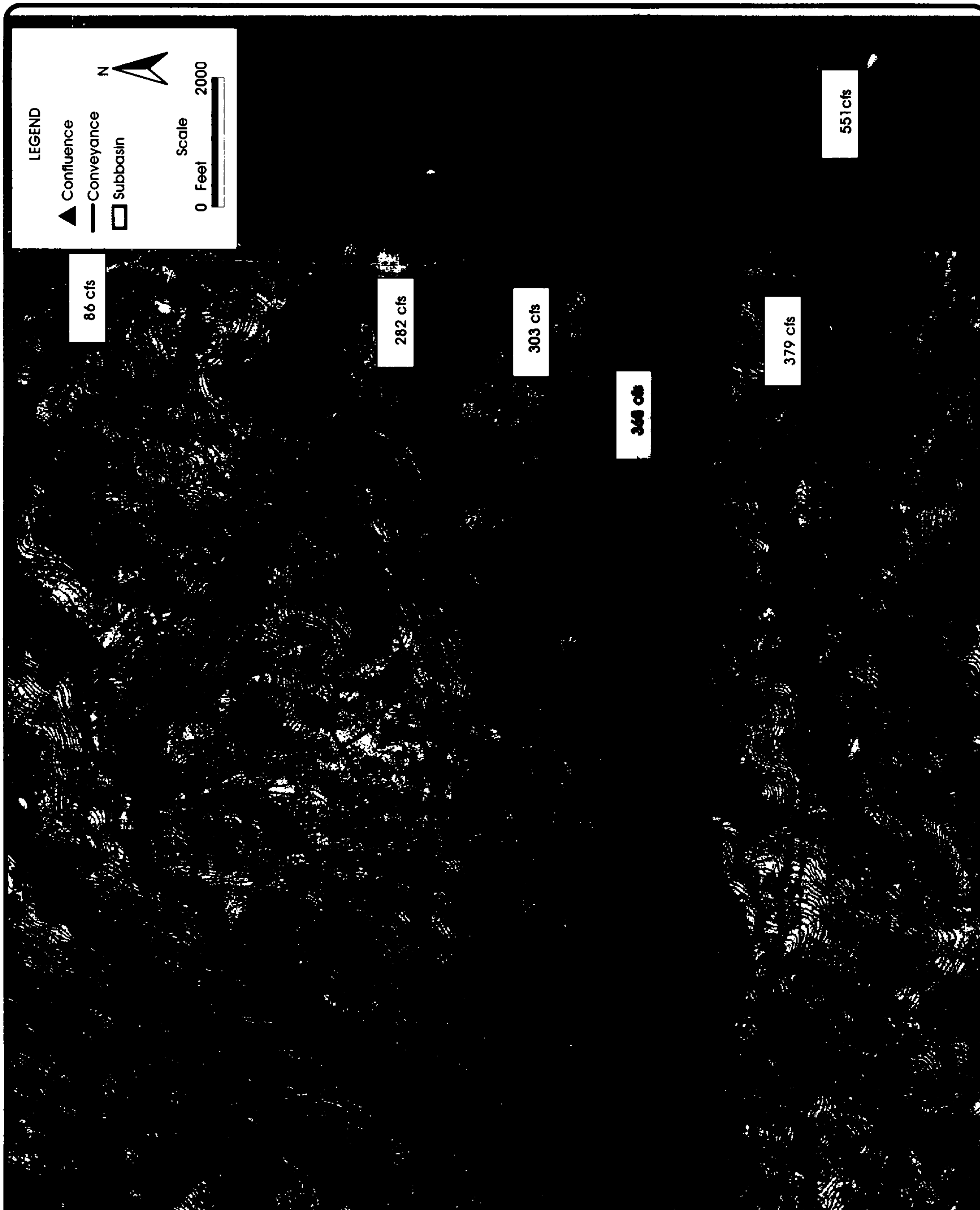
The magnitude of the area tributary to the landfill site is large enough to warrant the use of a reduction of the precipitation values because the likelihood of the full amount hitting the whole region decreases with an increase of tributary area. The factor was based on the Salt Lake City Hydrology Manual. According to the manual, a 24-hour event has an Areal Reduction Factor of:

$$ARF = .01*(100-2*Area^{.46}) \text{ where the Area} = 3.68 \text{ mi}^2$$

$$ARF = 0.96$$

This reduction factor was applied to each sub-basin's precipitation value.

<u>Peak Design Flows.</u> Hydrologic calculations presented above were used to generate peak design flows for each of the sub-basins and at various confluence points along the channels. Peak design flows are provided on Figure V-3.



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OFF-SITE MODEL RESULTS

FIGURE V-3

On-Site Run-Off Storm Water

Storm water will need to be conveyed off the landfill facility in order to protect the integrity of the closure cap.

Methodology. Delineation of the sub-basins, shown in Figure V-4, was based on the cell closure cap design. Each basin will drain into a channel which will convey the runoff to a down spout.

A curve number was determined based on the hydrologic soil type, Type B, found at the facility because native soils are going to be used for cover. The cover type was assumed to be similar to a dirt road. The cover conditions were combined with the hydrologic soil type to produce a curve number based on Table 2-2a of Technical Release 55. A curve number of 82 was applied to all on-site sub-basins.

The lag times, defined as the time to the hydrograph peak, were calculated by using thetime of concentration and the equation $T_l = 0.6T_c$. The time of concentration was calculated using the criteria found in Worksheet 3 in TR-55 "Urban Hydrology of Small Watersheds".

The SCS Type II Distribution was used with the 100-year 24-hour storm. The rainfall amount was taken from the "Point Precipitation Frequency Estimates from NOAA Atlas 14" associated with the facility elevation which is lower than the elevation used for the precipitation amount from the Lakeside Mountains. The value for a 100 year - 24 hour event for the facility is 2.52 inches.

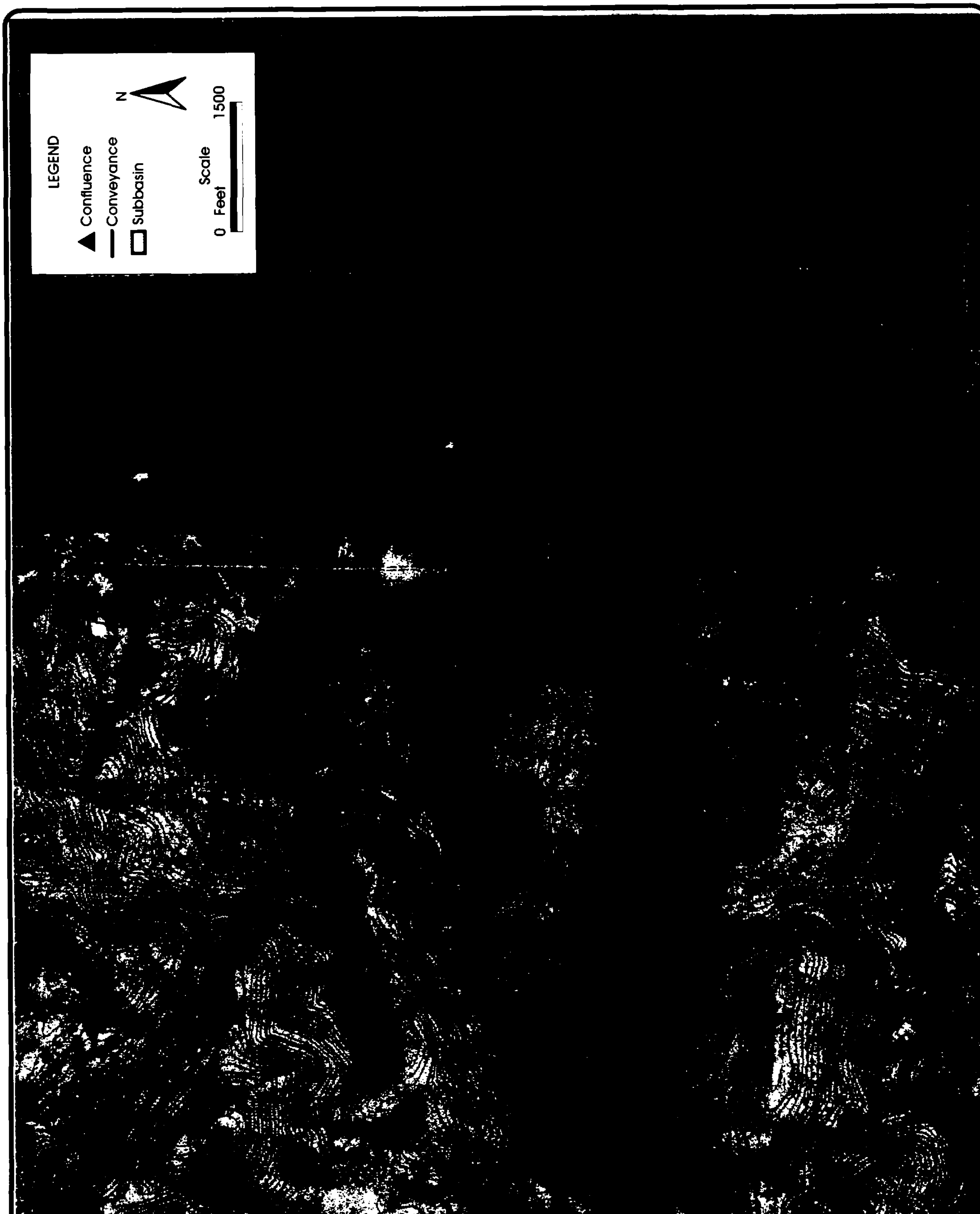
Peak Design Flows. The hydrologic analysis presented above was used to generate peak design flows for each of the sub-basins for the closure cap and for the downspout piping located at points along the east side of the closure cap as shown in Figure V-5.

HYDRAULIC DESIGN OF CHANNELS

The design peak flows for the channel segments provided in Table V-1 were used to design the drainage channels. The channels were designed with a 2.5H:1V side slope using the slope of the mountainside (or the western side fo the channel away from the closure cap) and a 4H:1V slope resulting from the closure cap slope.

A drainage channel with a bottom width of 15 feet will be constructed along the western perimeter of the closure cap to collect and convey storm water around the facility. Because the channel slopes vary from from 0.25% to 15% and the flows vary from 86 cfs to 521 cfs, the depth requirement and riprap design will vary along the channel reaches. Riprap D_{50} requirements for each segment are summarized in Table V-1. The minimum depth requirements include 1 foot of freeboard.

The landfill cells will be opened up gradually from the east to the west, therefore, construction of the drainage channels will not be required until landfill construction extendsto the Lakeside Mountains forming the west side of the landfill. Temporary run-on diversion berms will be constructed along the west side of constructed portions of the landfill until the landfill area ties into the Lakeside Mountains and construction of the drainage channels becomes necessary. These berms will prevent run-on water from the Lakeside Mountains and the west area of the facility from entering open landfill areas.



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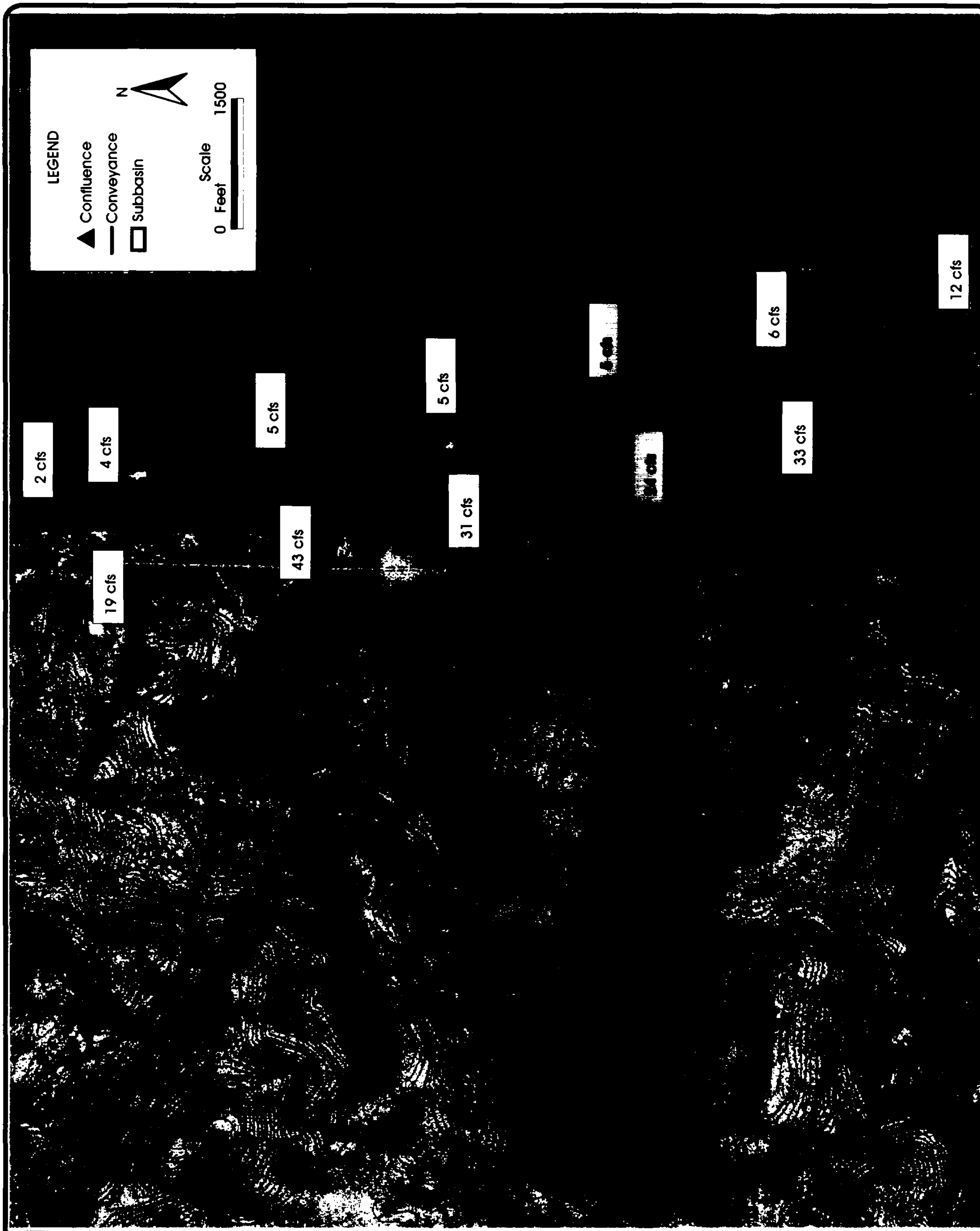
ON-SITE HYDROLOGY MODEL

FIGURE
V-4



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ON-SITE MODEL RESULTS

FIGURE V-5

TABLE V-1
RIPRAP DESIGN

Channel Segment	Slope	Peak Design Flow (CFS)	Rip Rap D_{50} Size (ft)	Rip Rap D_{50} Size (in)	Min Depth (ft)
Channel 1-A	0.25%	303	0.33	4	4.2
Channel 1-B	1.00%	303	1.0	12	4.0
Channel 1-C	5.00%	368	2.5	30	4.0
Channel 1-D	2.00%	368	1.75	21	4.2
Channel 1-E	0.25%	379	0.33	4	4.7
Channel 1-F	5.00%	551	2.75	33	4.8
Channel 1-G	1.00%	551	1.17	14	5.2
Channel 2-A	0.25%	63	0.25	3	2.5
Channel 2-B	2.00%	86	1.0	12	2.6
Channel 2-C	5.00%	86	1.75	21	2.5
Channel 2-D	15.00%	86	2.5	30	2.4
Channel 2-E	1.50%	86	0.75	9	2.6

DOWNSPOUT DESIGN

Hydrologic calculations presented above were used to generate the combined peak design flows. To maintain consistency in design and construction, the highest combined peak flows were used for design of the downspouts. Design is based on a combined peak flow of 12 cfs from the benches along the south 4H:1V slopes of the cap, 43 cfs from each drainage area on top of the cap, and 6 cfs for the benches along the eastern 4H:1V slopes.

Downspout pipe sizes were determined using inlet control conditions and selecting the size and head water depth requirement from "Hydraulic Charts for the Selection of Highway Culverts" published by the U.S. Department of Transportation. Inlet control conditions were assumed because critical flow will always exist in the piping on the 4H:1V slopes and the elevation differences between the inlet and outlet ends of the downspout pipes will not allow for outlet conditions to control.

Downspout pipe sizes and head water depth requirements for the south benches, top of cap and eastern benches are:

1. South benches require 24-inch diameter pipe with 2 feet of headwater depth
2. Downspout pipes from the top of the cap require two 24-inch diameter pipes in parallel with 3 feet of headwater depth.
3. East benches require 15-inch diameter pipe with 2 feet of headwater depth.

The headwater depth requirements are provided with the inlet boxes below the grating with the additional depth and freeboard provided by the grating and the ditches and berm heights above the grating.

EROSION PROTECTION

Long term options to provide erosion protection generally consist of establishing vegetation, or by placing a stone mulch, or a combination of both. Procedures presented in "Erosion and Sedimentation in Utah - A Guide for Control" published by the Utah Water Research Laboratory were used to determine requirements for vegetative and stone mulch erosion control measures. Calculations show that the density of the vegetative cover should be 93 percent and the minimum thickness of the stone mulch is 3 inches. Stone mulch generally consists of a well graded stone or gravel with the largest size being approximately equal to the required stone mulch thickness.

DETENTION

All stormwater will be routed into the borrow excavation area of the property directly east of the landfill site that will also be used for storm water management. The off-site runoff will continue in open channels and pipes (primarily under facility roads and for the inlet to the detention area) to the detention area. Flow from the downspout pipes will either continue to be conveyed to the detention area in pipes, open channels, or a combination of both. Upon completion, this excavation will be approximately 20 feet deep or more with a surface area of approximately 600 acres. A 24-inch diameter storm drain pipe will be placed under the railroad and road at the eastern end of the excavation with an inlet flow line elevation of 4220 to provide an outlet for storm water from the detention basin. Using the Army Corps of Engineers HEC-1 model to simulate routing of storm water through the basin shows a maximum headwater depth on the storm drainage pipe of about 3 feet. This headwater depth will be temporary as the outlet to the basin will allow the ponded water to drain and empty the basin to the flow line elevation of the outlet.

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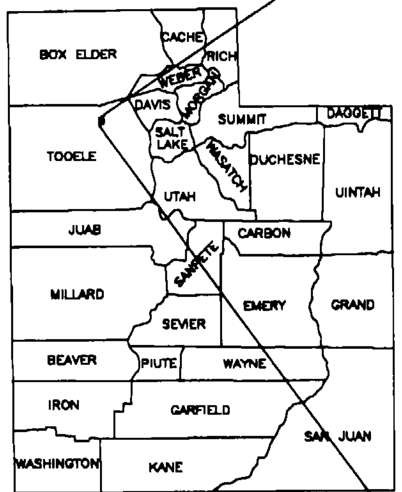
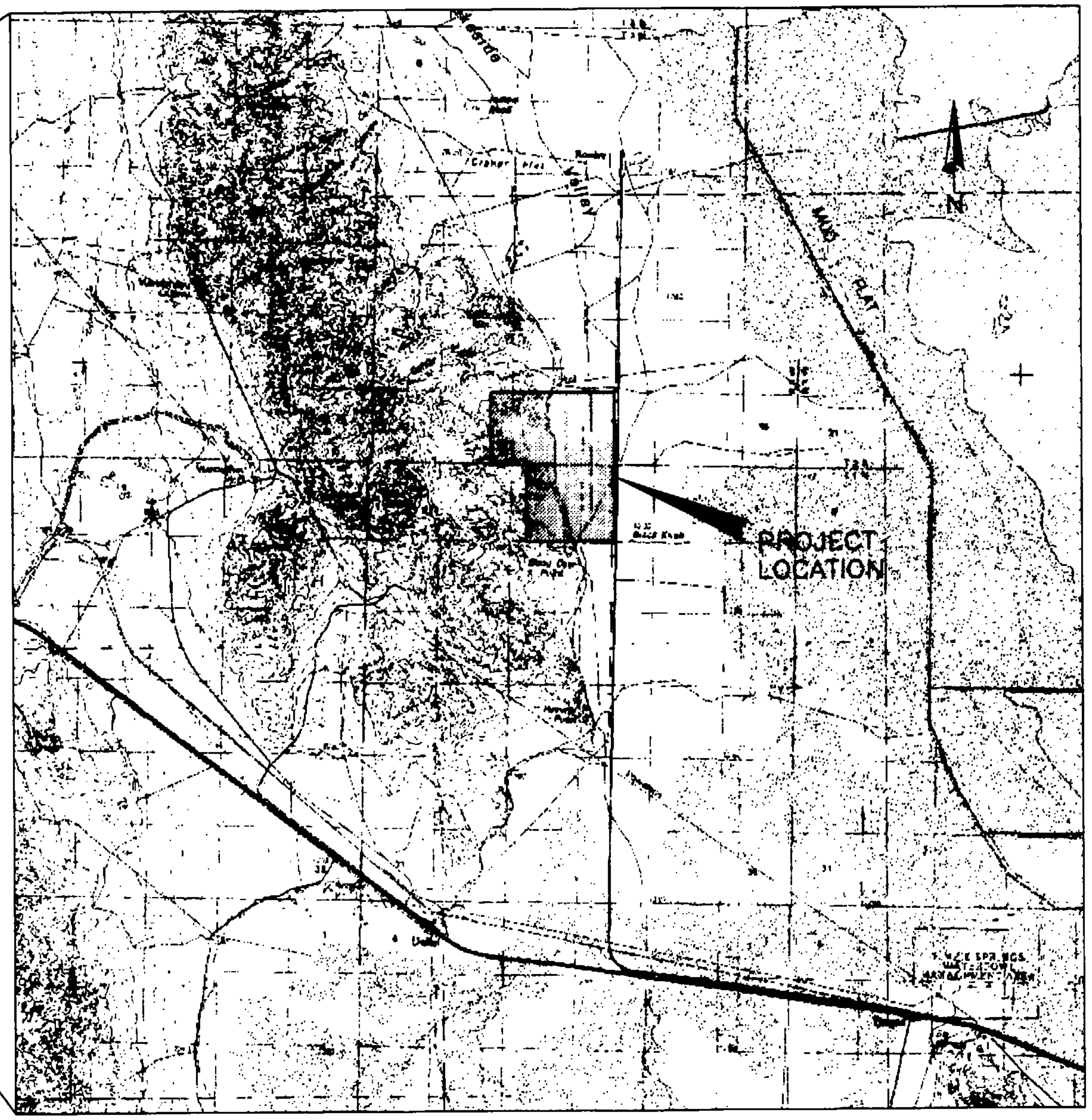
APPENDIX A

PERMIT DESIGN DRAWINGS

WASATCH REGIONAL LANDFILL FACILITY



STATE OF UTAH



PROJECT LOCATION

INDEX OF DRAWINGS

SHEET NO.	TITLE
1.	COVER SHEET
2.	EXISTING SITE TOPOGRAPHY
3.	CELL LCRS & SUPPORT FACILITIES PLAN
4.	CLOSURE SITE PLAN
5.	OVERALL CELL SECTIONS
6.	PHASE 1A PLAN & SECTIONS
7.	SUMP PLAN & SECTIONS
8.	LEACHATE WITHDRAWL PIPE SECTIONS
9.	LEACHATE WITHDRAWL SYSTEM DETAILS
10.	TYPICAL LINER SYSTEM SECTIONS & DETAILS
11.	CLOSURE CAP DETAILS
12.	DOWNSPOUT PLAN & PROFILE
13.	GROUND WATER INTERCEPTOR & STORM WATER BASIN SECTIONS
14.	GROUND WATER INTERCEPTOR & STORM WATER BASIN OUTLET SECTIONS
15.	LEACHATE EVAPORATION POND DETAILS
16.	FACILITY ACCESS ROAD

REGISTERED PROFESSIONAL ENGINEER
12-27-04
NO. 169141
KENT C. STAHELI
STATE OF UTAH

ENGINEERS:

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6771 SOUTH 900 EAST
MIDVALE, UTAH 84047
(801) 566-5599

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS
600 WEST SANDY PARK WAY
SANDY, UTAH 84070
(801) 566-6399

FILE NAME: 113\30.100\CADFILES\PERMIT DWGS\COVER.DWG
FILE DATE: 12.20.2004 13:47:49 (JDB)

7/04

HANSEN ALLEN & LUCE inc ENGINEERS

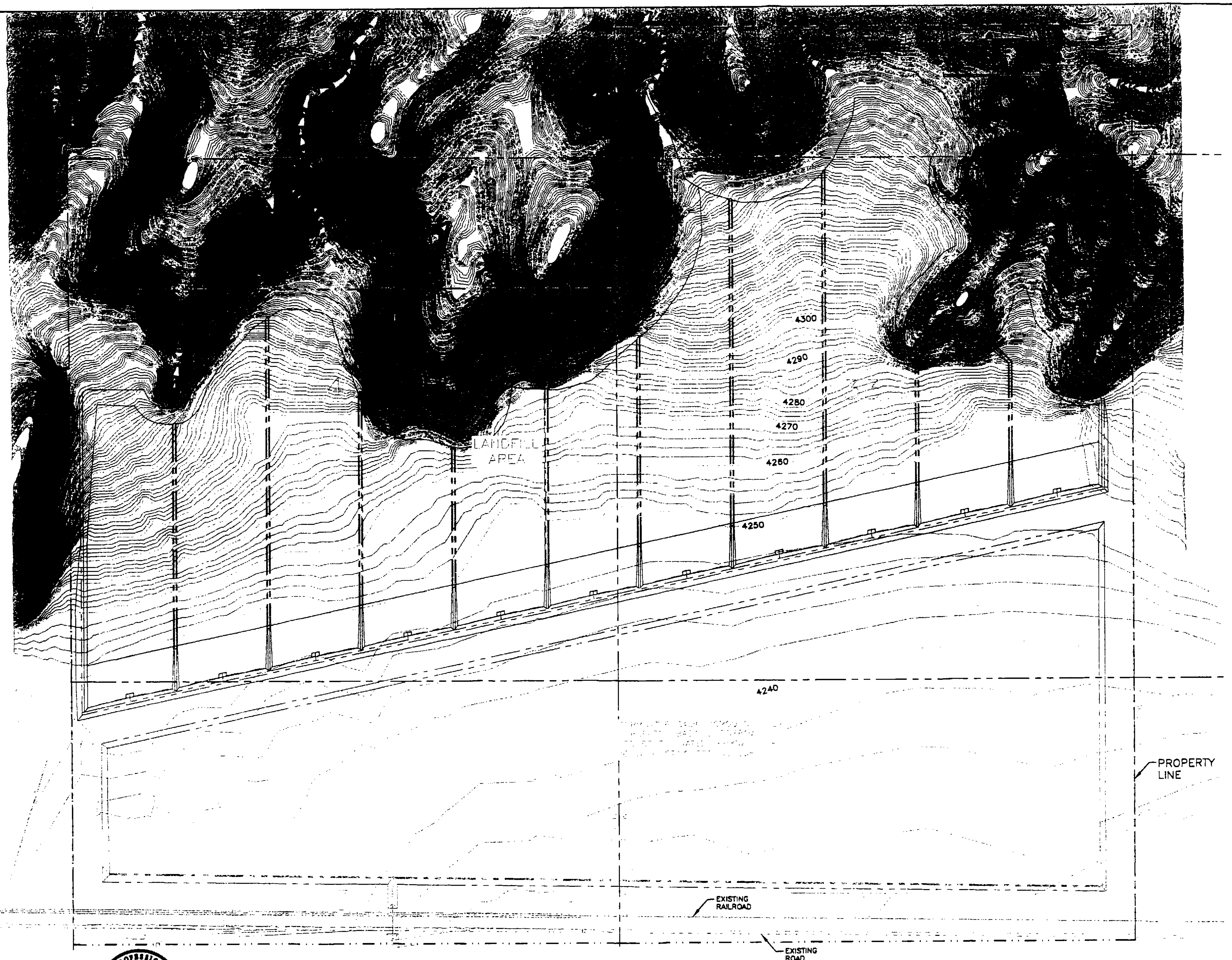
	DESIGNED MPW, KCS	3		
	DRAFTED CAH	2		
	CHECKED KCS	1		
PROJECT ENGINEER	DATE DECEMBER 2004	NO.	DATE	REVISIONS BY APVD.

SCALE NOT TO SCALE

WASATCH REGIONAL

WASATCH REGIONAL LANDFILL FACILITY
COVER SHEET

SHEET 1
113-30-100



SECTION & DETAIL IDENTIFICATION

SECTION IDENTIFICATION

SECTION CUT ON DRAWING NO. 6 AND SHOWN ON DRAWING NO. 8 ON DRAWING NO. 6 THIS SECTION IS REFERENCED AS:

2 / 8 — SECTION NUMBER; DRAWING ON WHICH SECTION APPEARS

ON DRAWING NO. 8, THIS SECTION IS IDENTIFIED AS:

SECTION 2 / 6 — SECTION NUMBER; DRAWING FROM WHICH SECTION WAS TAKEN

DETAIL IDENTIFICATION

DETAIL CALL-OUT ON DRAWING NO. 6 AND SHOWN ON DRAWING NO. 8 ON DRAWING NO. 6 THIS DETAIL IS REFERENCED AS:

B / 8 — DETAIL LETTER; DRAWING ON WHICH DETAIL APPEARS

ON DRAWING NO. 8, THIS DETAIL IS IDENTIFIED AS:

DETAIL B / 6 — DETAIL LETTER; DRAWING FROM WHICH DETAIL WAS TAKEN

NOTES:

1. IF SECTION AND DETAILS ARE SHOWN ON THE SAME DRAWING AS SECTION CUTS AND SECTION OR DETAIL CALL-OUTS DRAWING NUMBER IS REPLACED BY A LINE.
2. DETAIL LETTERS "I" AND "O" NOT USED.

TABLE OF ABBREVIATIONS

℄	CENTER LINE
CPE	CORRUGATED POLYETHYLENE
DIA.	DIAMETER
EL.	ELEVATION
FL	FLOW LINE
HDPE	HIGH DENSITY POLYETHYLENE
ID	INSIDE DIAMETER
INV EL.	INVERT ELEVATION
MAX.	MAXIMUM
MIN.	MINIMUM
N.T.S.	NOT TO SCALE
PCPE	PERFORATED CORRUGATED POLYETHYLENE PIPE
SDR	STANDARD DIMENSION RATIO
TYP.	TYPICAL

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HANSEN ALLEN & LUCE INC. ENGINEERS

PROJECT ENGINEER

DESIGNED MPW, KCS	3				
DRAFTED CAH	2				
CHECKED KCS	1				
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY	APVD.

SCALE: AS SHOWN

WASATCH REGIONAL

WASATCH REGIONAL LANDFILL FACILITY

EXISTING SITE TOPOGRAPHY

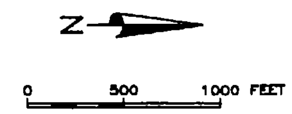
SHEET 2

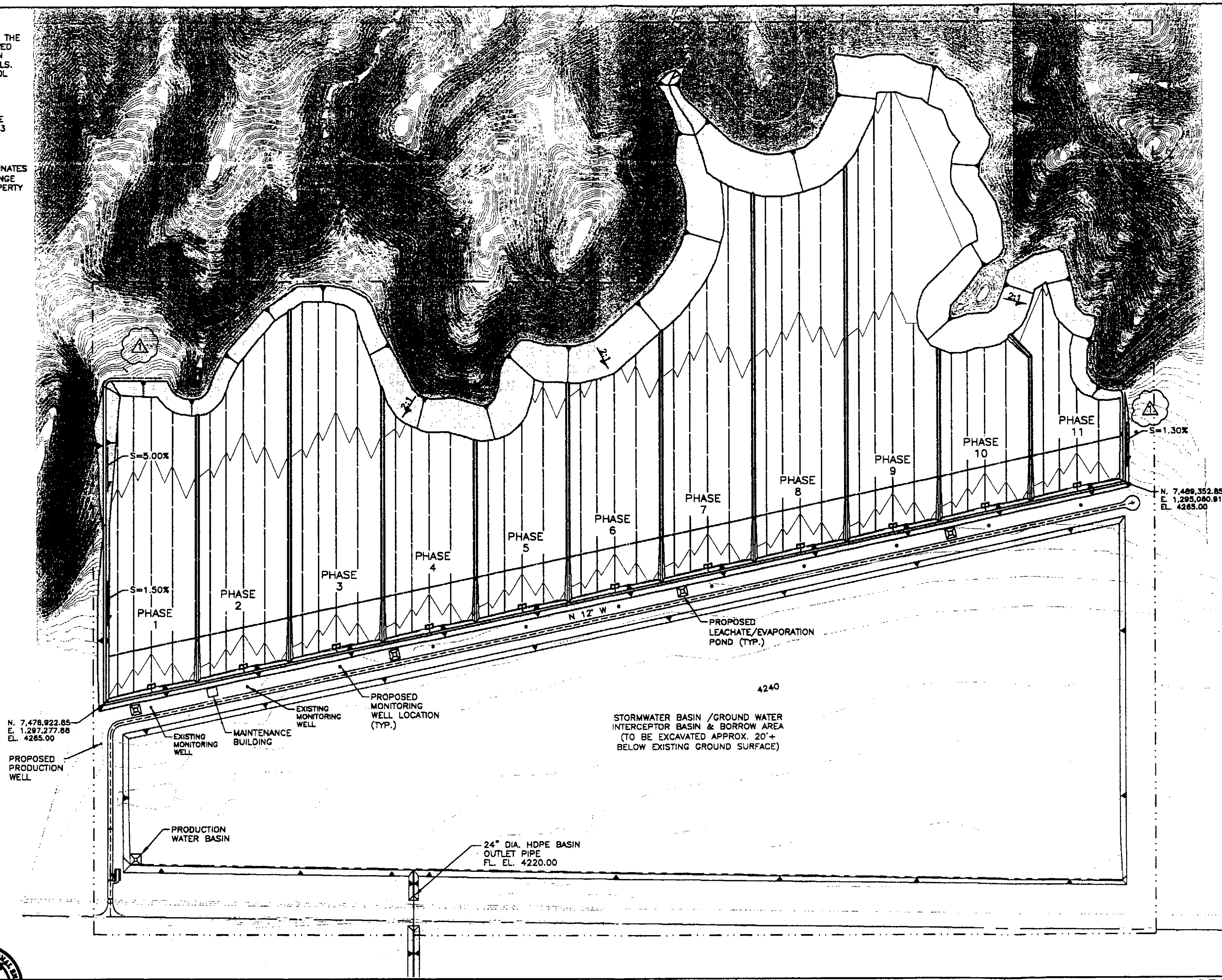
113-30-100

7/04

NOTES:

1. ROCK OUTCROPPING ALONG THE WEST HILLSIDE MAY BE MINED FOR RIPRAP AND OTHER ON SLOPE PROTECTIVE MATERIALS. CELL AND RUN-ON CONTROL SYSTEM DESIGN WILL BE MODIFIED AS REQUIRED TO MATCH MINED SURFACES.
2. COORDINATES PROVIDED ARE BASED ON MODIFIED NAD 83 COORDINATE SYSTEM STATE PLANE (GROUND)
3. ACTUAL LOCATIONS (COORDINATES AND ELEVATIONS) MAY CHANGE AFTER CONDUCTING A PROPERTY SURVEY AND ACCURATE TOPOGRAPHIC MAPPING.





PHASES	
PHASE	AREA
1	67.20 ACRES
2	74.08 ACRES
3	81.14 ACRES
4	53.01 ACRES
5	55.76 ACRES
6	58.78 ACRES
7	95.37 ACRES
8	88.10 ACRES
9	133.95 ACRES
10	41.12 ACRES
11	44.60 ACRES
TOTAL = 793.11 ACRES	

FILE 113\30.100\CADFILES\PERMIT DWGS\SITE PLAN__R1.DWG
FILE DATE: 5.2.2005 11:18:10 (CAH)

7/04

HANSEN ALLEN & LUCE INC ENGINEERS

PROJECT ENGINEER

DESIGNED MPW, KCS	3					
DRAFTED CAH	2					
CHECKED KCS	1	05/2005	ADDED PROPOSED MONITORING WELL & REMOVED NOTE	CAH	KCS	
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY	APVD.	

SCALE AS SHOWN

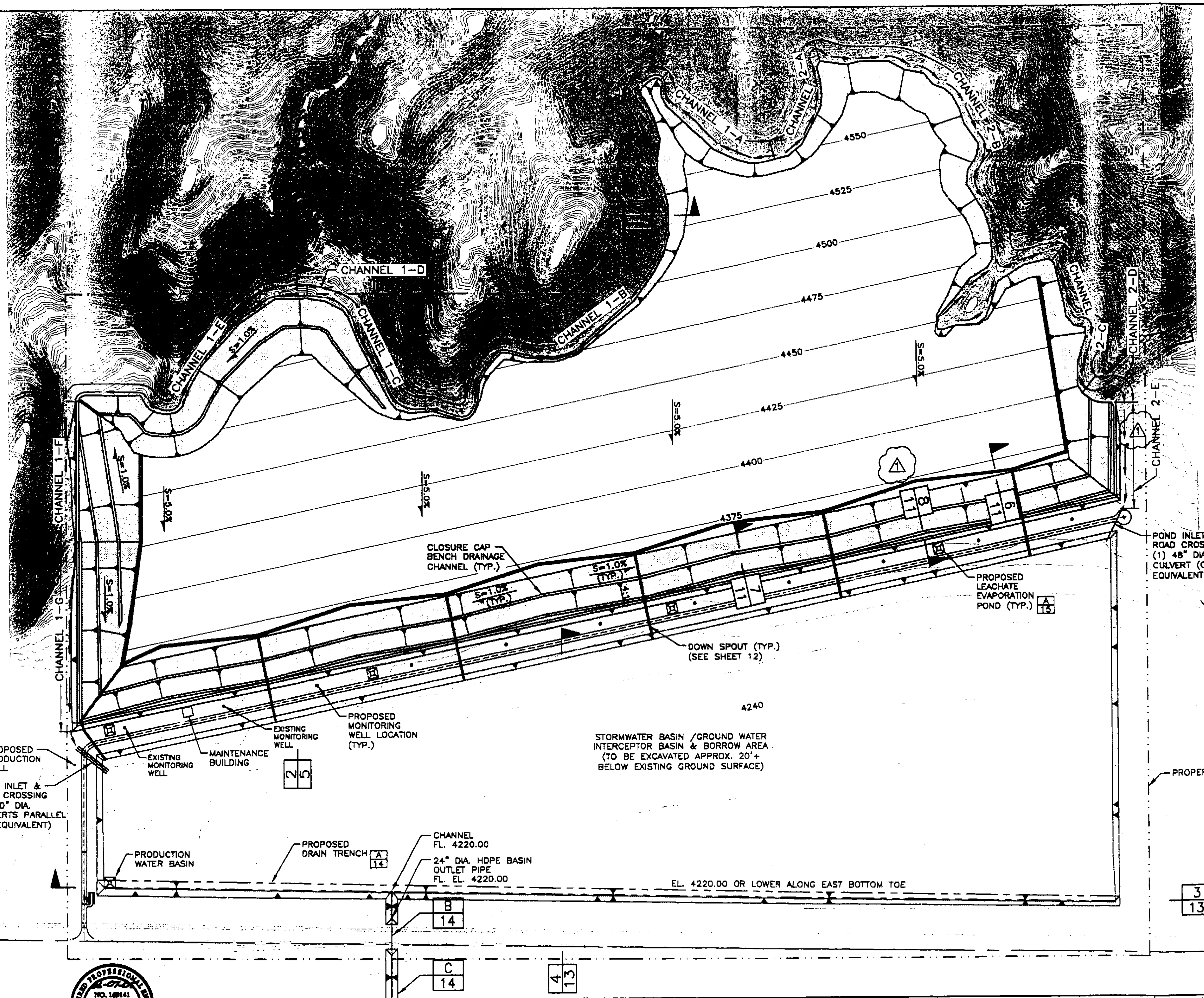
WASATCH REGIONAL

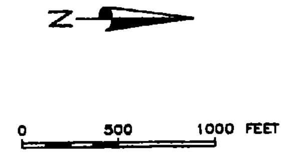
WASATCH REGIONAL LANDFILL FACILITY

CELL LCRS & SUPPORT FACILITIES PLAN

SHEET 3

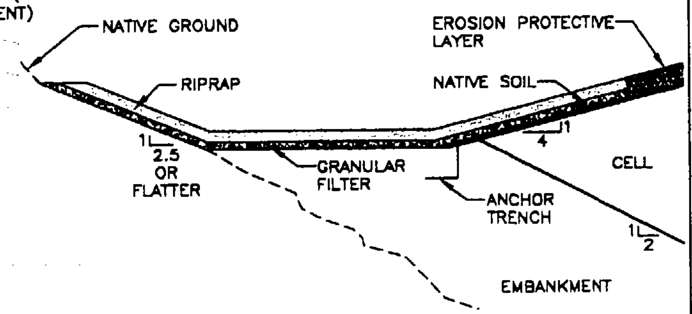
113-30-100





RUN-ON CHANNEL DESIGN TABLE

CHANNEL SEGMENT	SLOPE	PEAK DESIGN FLOW (CFS)	RIPRAP D50 (FT)	MIN. DEPTH (FT)
CHANNEL 1-A	0.25%	303	0.33	4.23
CHANNEL 1-B	1.00%	303	1.0	4.03
CHANNEL 1-C	5.00%	368	2.5	4.02
CHANNEL 1-D	2.00%	368	1.75	4.19
CHANNEL 1-E	0.25%	379	0.33	4.67
CHANNEL 1-F	5.00%	551	2.75	4.80
CHANNEL 1-G	1.00%	551	1.17	5.20
CHANNEL 2-A	0.25%	86	0.25	2.51
CHANNEL 2-B	2.00%	86	1.0	2.54
CHANNEL 2-C	5.00%	86	1.75	2.46
CHANNEL 2-D	15.00%	86	2.5	2.36
CHANNEL 2-E	1.50%	86	0.75	2.60



TYPICAL SECTION 5 / –
N.T.S.

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DESIGNED MPW, KCS	3			
DRAFTED JDB	2			
CHECKED KCS	1	05/2005	ADDED PROPOSED MONITORING WELL & REMOVED NOTE	CAH / KCS
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY / APVD.

HANSEN ALLEN & LUCE INC ENGINEERS — PROJECT ENGINEER

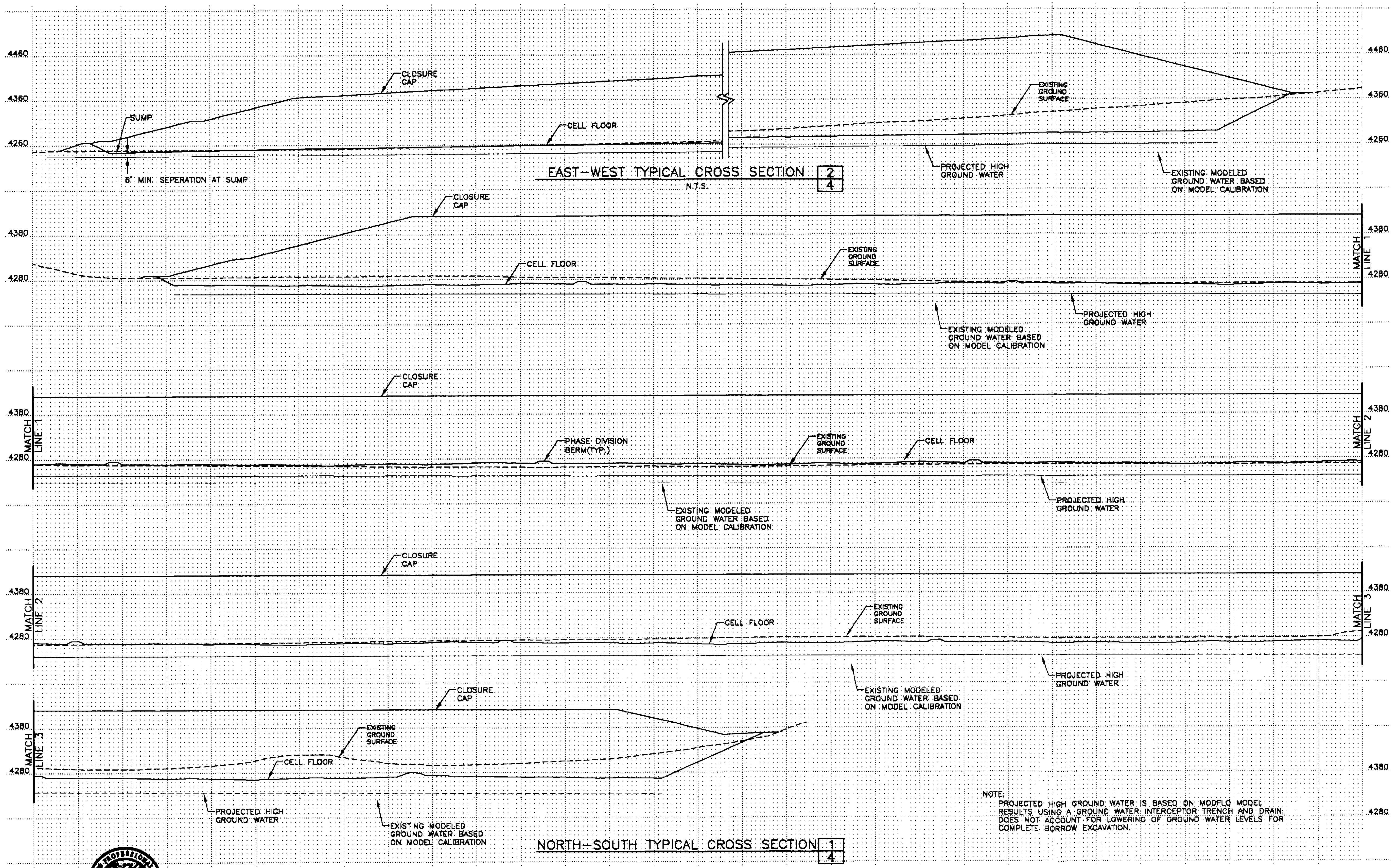
SCALE: AS SHOWN

WASATCH REGIONAL

WASATCH REGIONAL LANDFILL FACILITY
CLOSURE SITE PLAN

SHEET 4
113-30-100

7/04



HANSEN ALLEN & LUCE INC. ENGINEERS	PROJECT ENGINEER	DESIGNED MPW, KCS	3			SCALE 1" = 200'	WASATCH REGIONAL	WASATCH REGIONAL LANDFILL FACILITY
		DRAFTED DRB	2					OVERALL CELL SECTIONS
		CHECKED KCS	1					SHEET 5
		DATE DECEMBER 2004	NO.	DATE	REVISIONS / BY / APVD.			113-30-100

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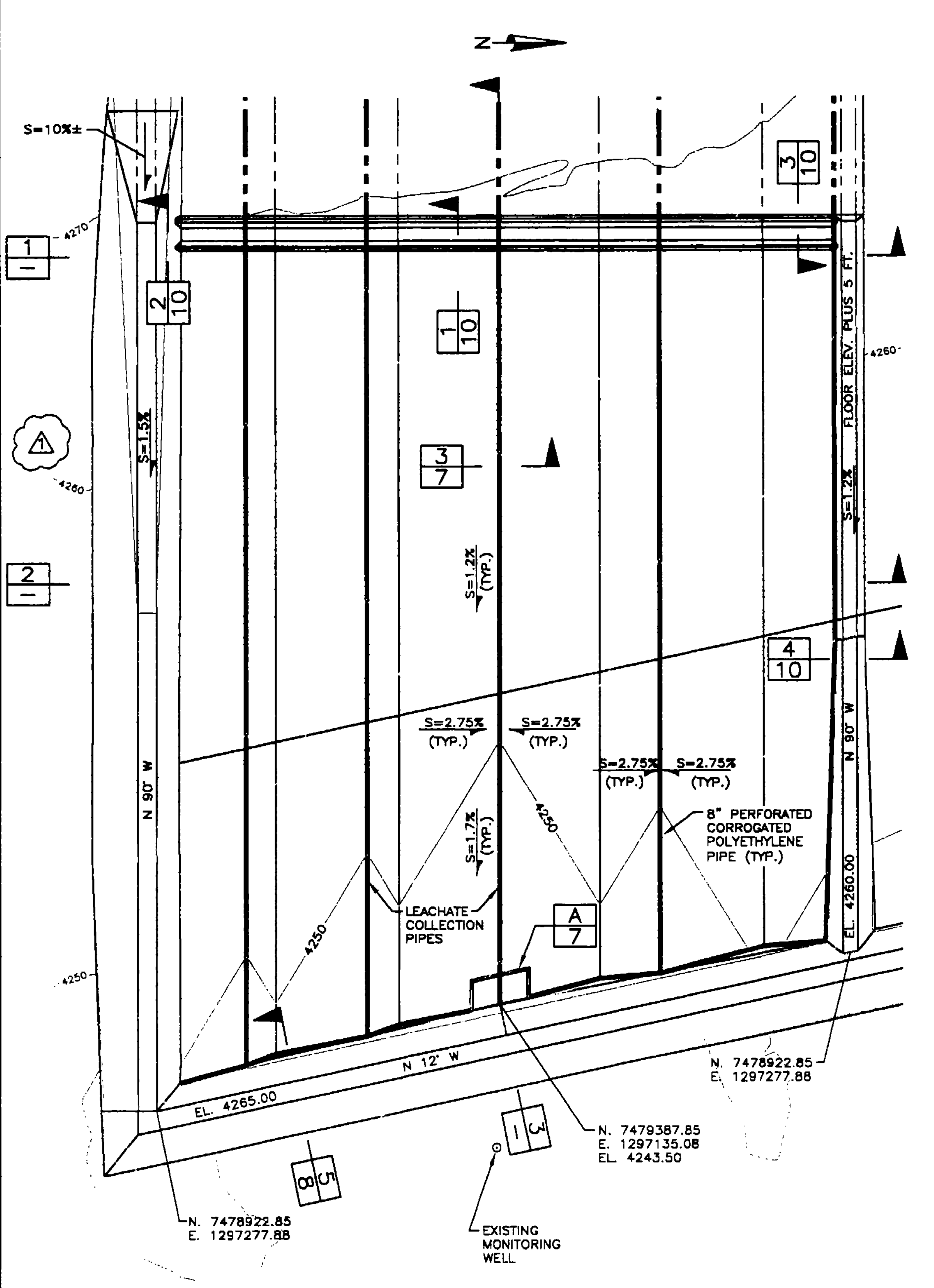
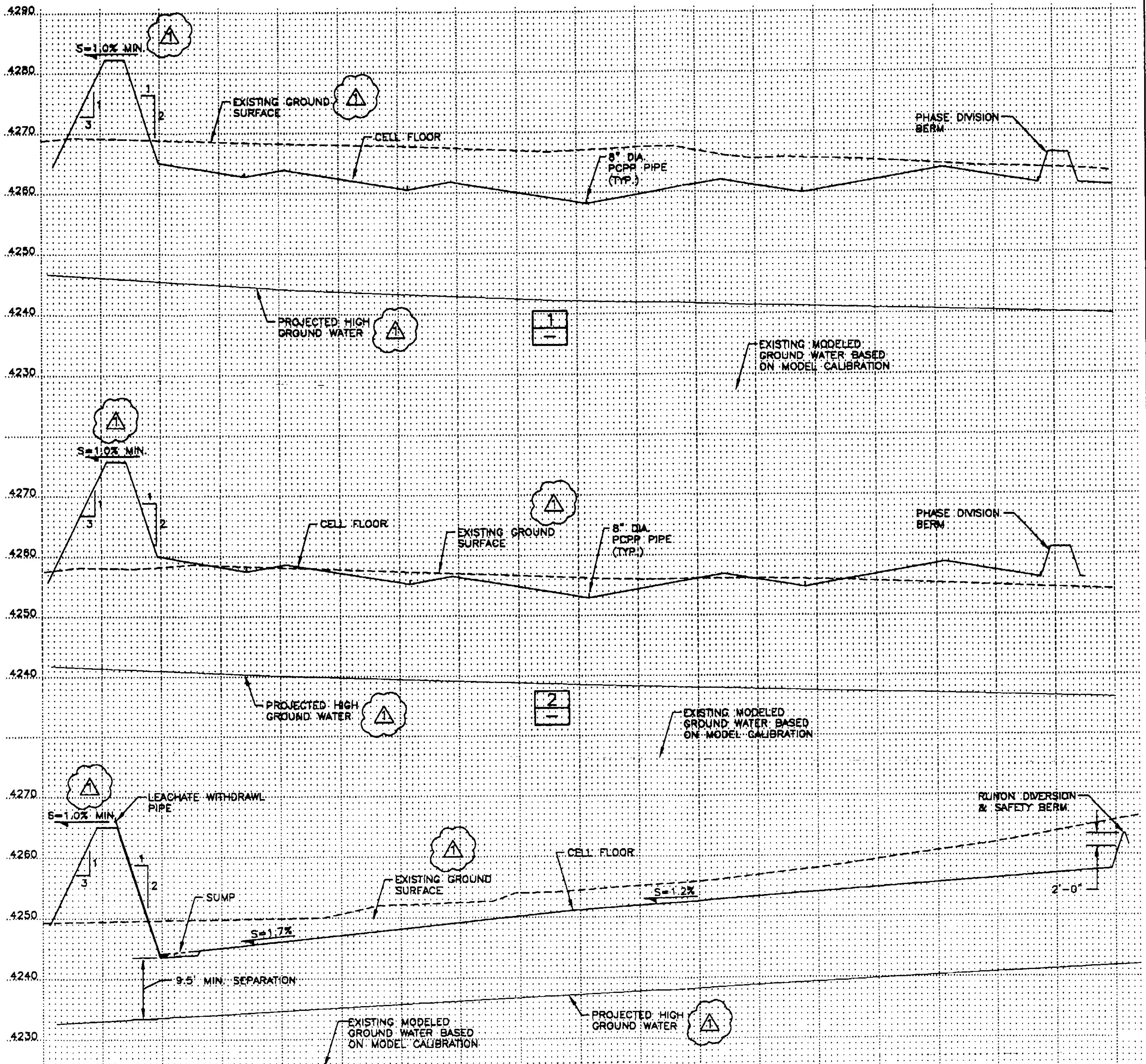
7/04



PHASE 1A PLAN

0 50 100 200



NOTE:
PROJECTED HIGH GROUND WATER IS BASED ON MODFLO MODEL RESULTS USING A GROUND WATER INTERCEPTOR TRENCH AND DRAIN. DOES NOT ACCOUNT FOR LOWERING OF GROUND WATER LEVELS FOR COMPLETE BORROW EXCAVATION.

TYPICAL CELL SECTIONS 3

HORIZONTAL 1" = 120'

VERTICAL 1" = 40'

HANSEN ALLEN & LUCE INC. ENGINEERS

KENT C. STAHELI NO. 189141 PROFESSIONAL ENGINEER STATE OF UTAH 5-07-05

DESIGNED MPW, KCS	3				
DRAFTED DRB	2				
CHECKED KCS	1	05/2005	ADDED SLOPE CALLOUTS, NEW CONTOURS, MODIFIES EXISTING GROUND SURFACE & PROJECTED HIGH WATER	CAH	KCS
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY	APVD.

PROJECT ENGINEER

SCALE AS SHOWN

WASATCH REGIONAL

WASATCH REGIONAL LANDFILL FACILITY

PHASE 1A PLAN & SECTIONS

SHEET 6

113-30-100

FILE 113\30.100\CADFILES\PERMIT DWGS\PHASE 1 PLAN__R1.DWG
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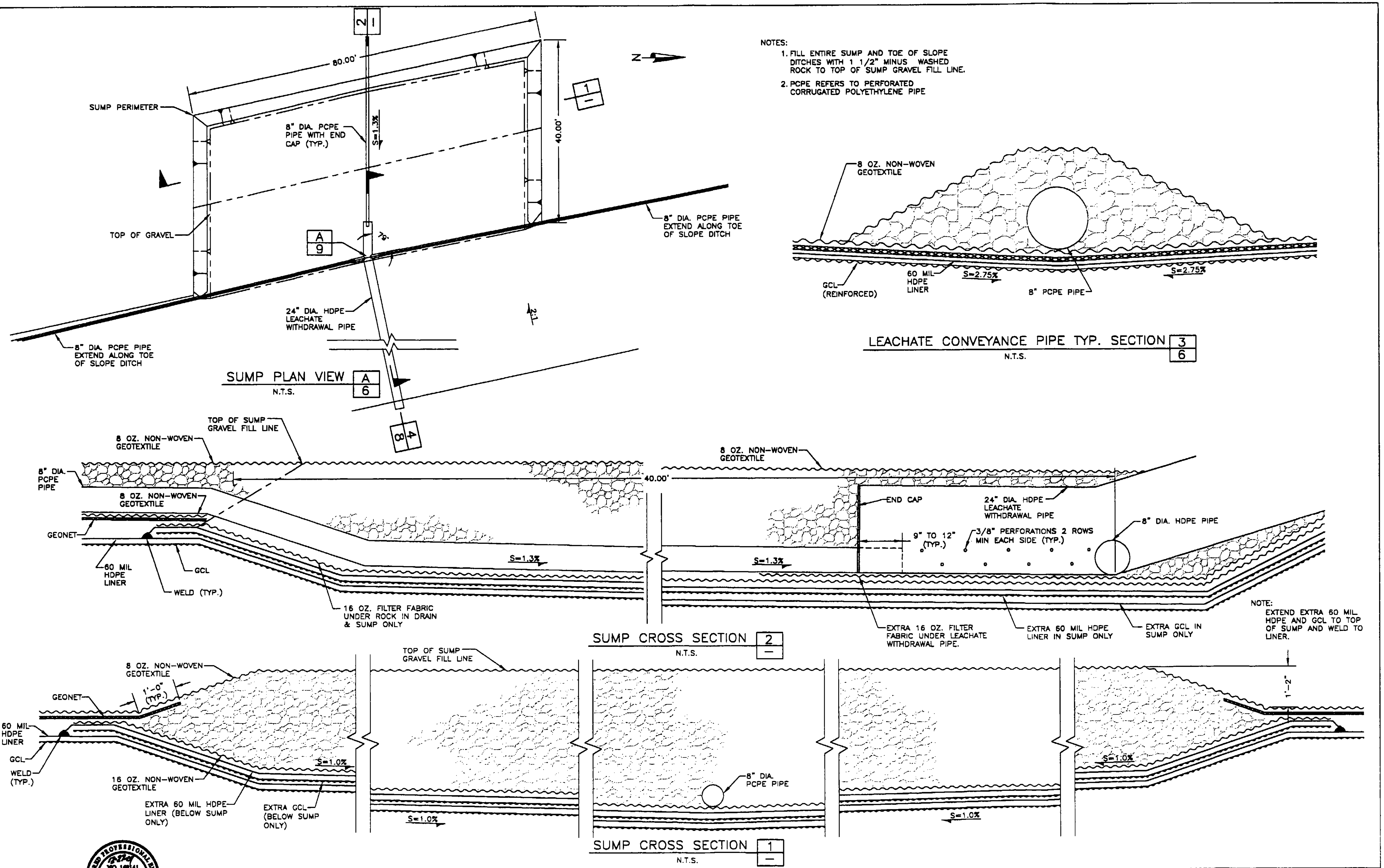
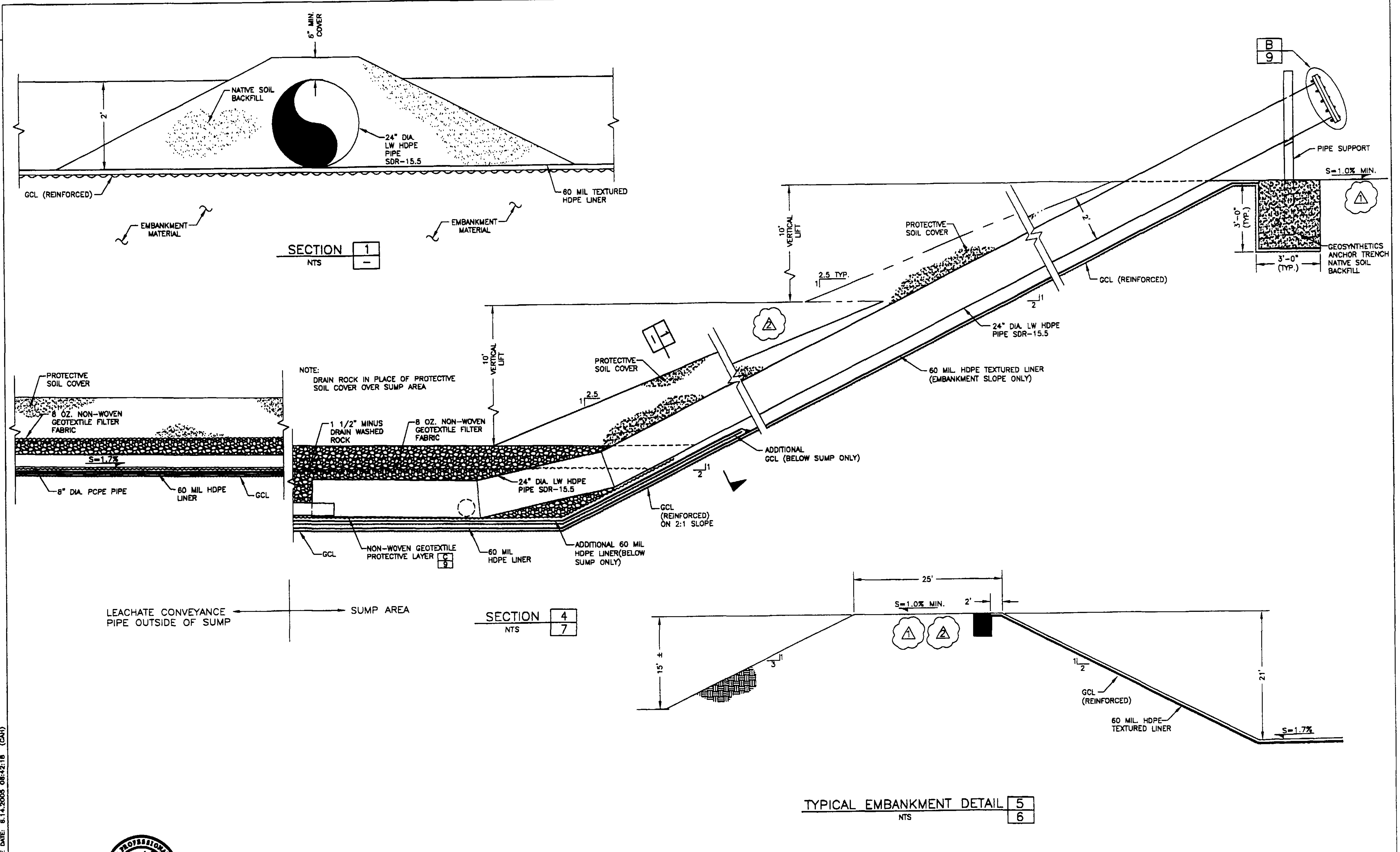
7/04



HANSEN ALLEN & LUCE INC ENGINEERS	DESIGNED MPW, KCS	3				SCALE 1" = 10'	WASATCH REGIONAL	WASATCH REGIONAL LANDFILL FACILITY	SHEET 7
	DRAFTED SDM	2						SUMP PLAN & SECTIONS	113-30-100
	CHECKED KCS	1							
PROJECT ENGINEER	DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY / APVD.				



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HANSEN ALLEN & LUCE INC ENGINEERS

PROJECT ENGINEER

DESIGNED MPW, KCS	3					
DRAFTED SDM	2	06/2005	CHANGED SLOPE DIRECTION & MODIFIED VIEW ON SECTION TO ILLUSTRATE LIFT HEIGHT	CAH	KCS	
CHECKED KCS	1	05/2005	REMOVED ROAD BASE & ADDED SLOPE CALLOUT	CAH	KCS	
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY	APVD.	

SCALE AS SHOWN

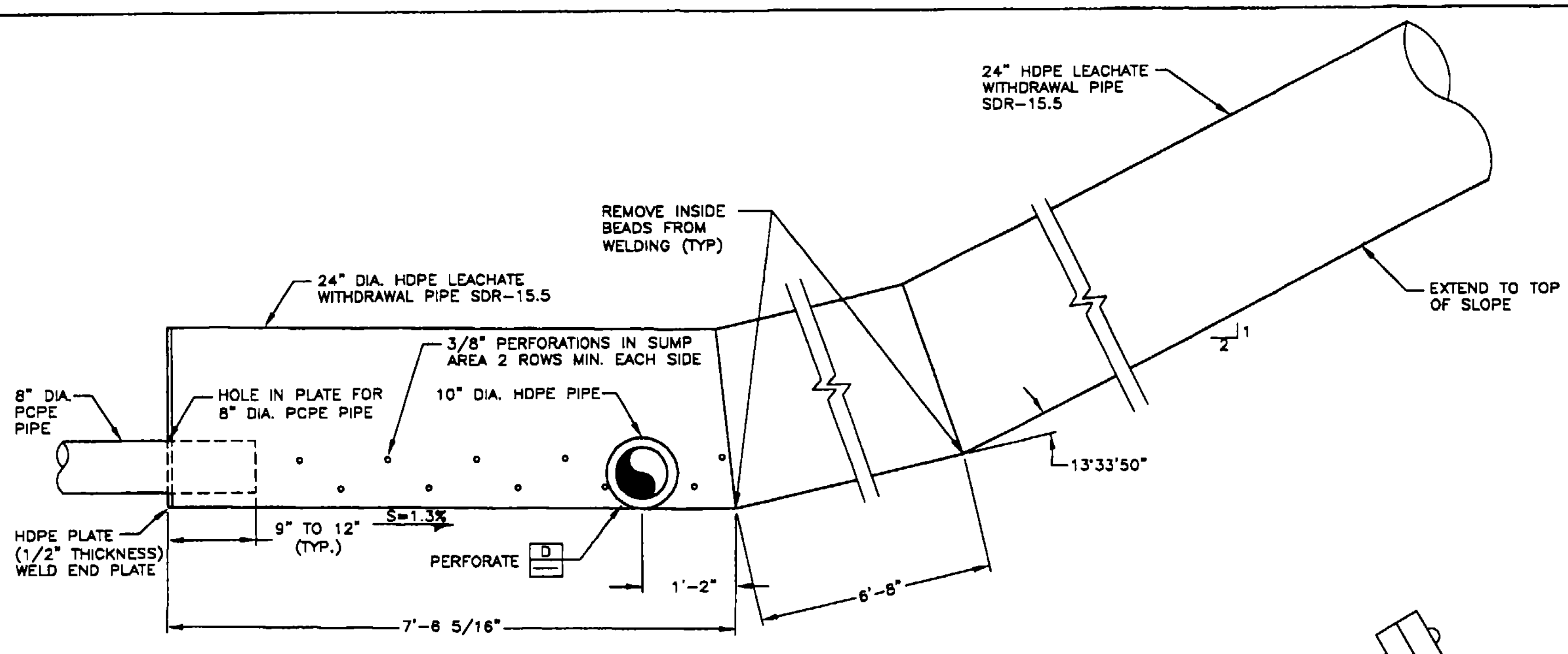
WASATCH REGIONAL

WASATCH REGIONAL LANDFILL FACILITY

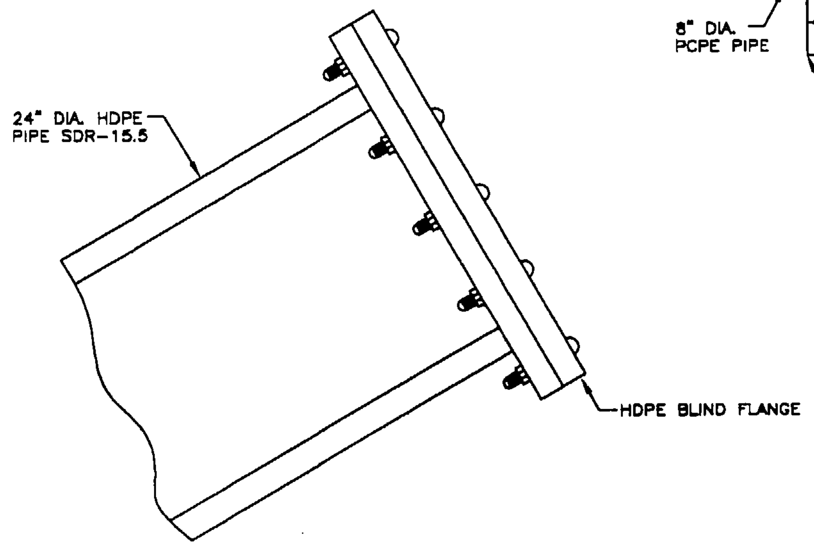
LEACHATE WITHDRAWAL PIPE SECTIONS

SHEET 8

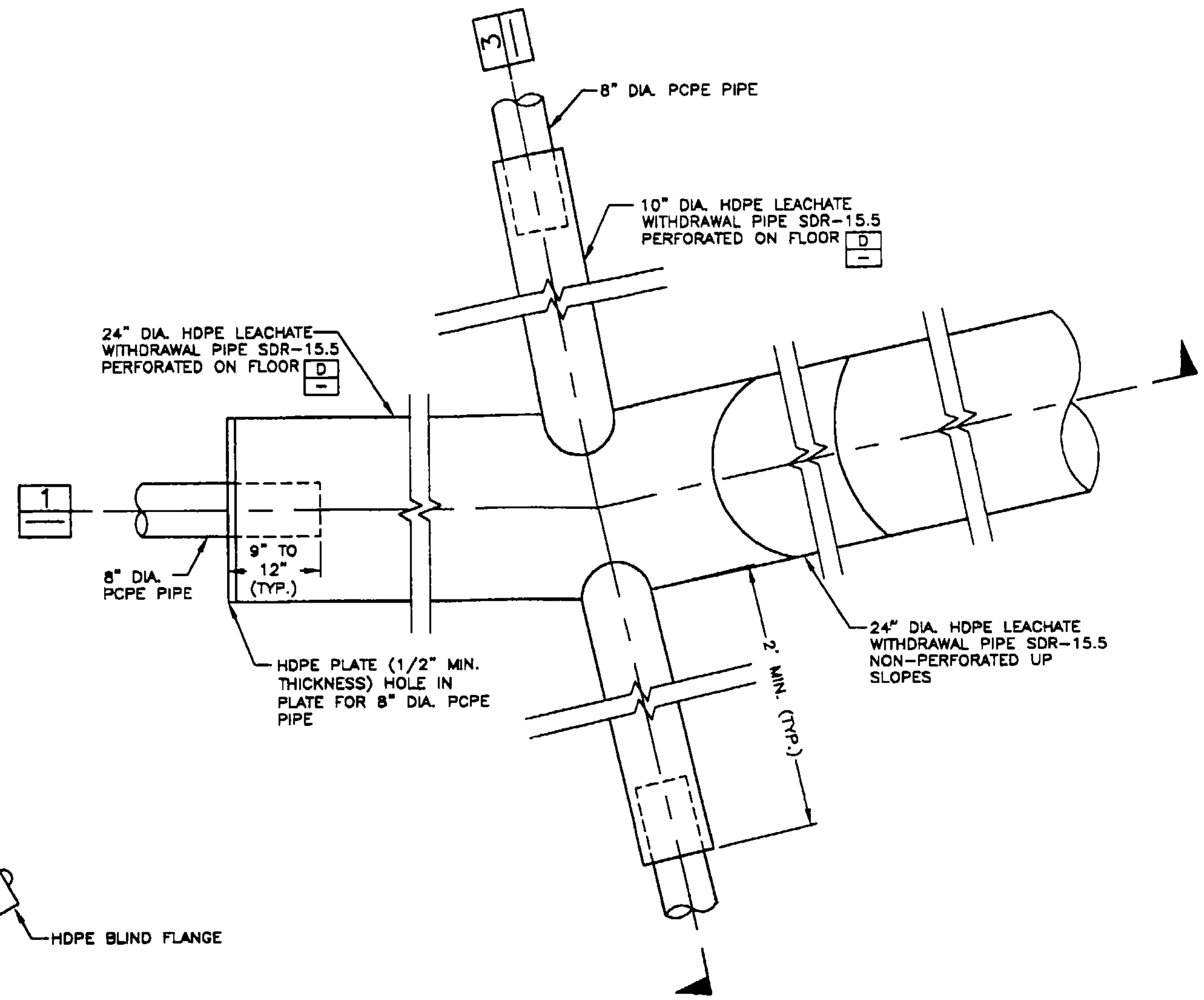
113-30-100



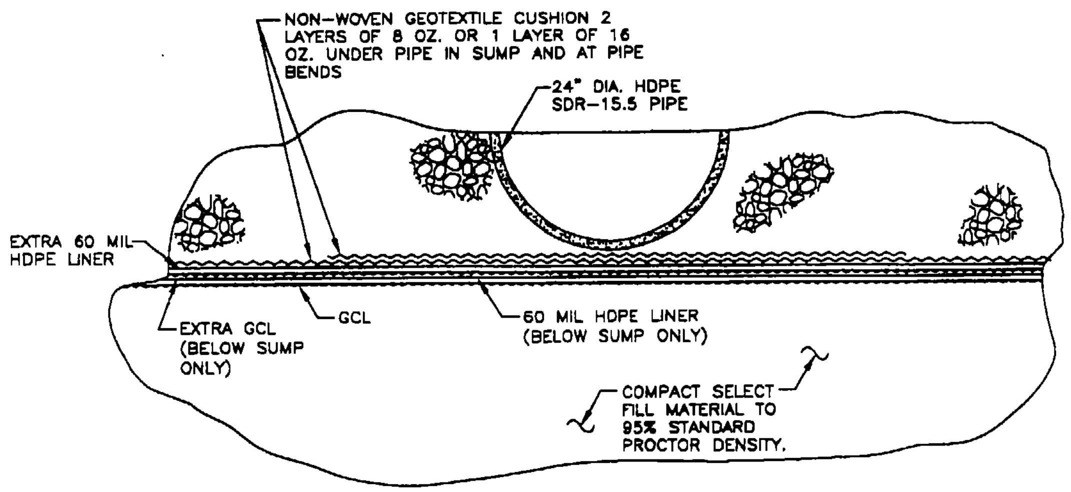
LEACHATE WITHDRAWAL PIPE SECTION 1
N.T.S.



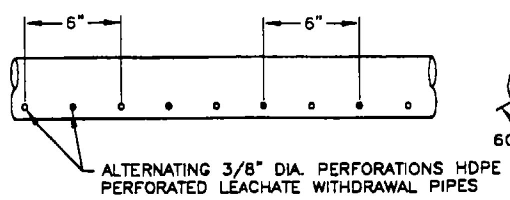
24" DIA. HDPE CAP DETAILS B 8
N.T.S.



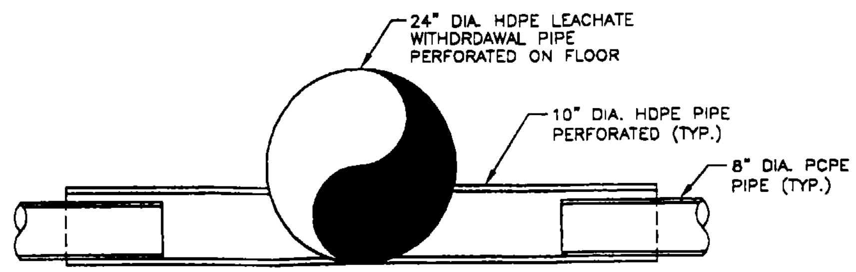
LEACHATE WITHDRAWAL PIPE DETAIL A 7
N.T.S.



DETAIL C 8
4" = 1'-0"



PERFORATION DETAIL D
N.T.S.



CROSS SECTION 3
N.T.S.

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HANSEN ALLEN & LUCE inc ENGINEERS

PROJECT ENGINEER

DESIGNED MPW, KCS	3				
DRAFTED CAH	2				
CHECKED KCS	1				
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY	APVD.

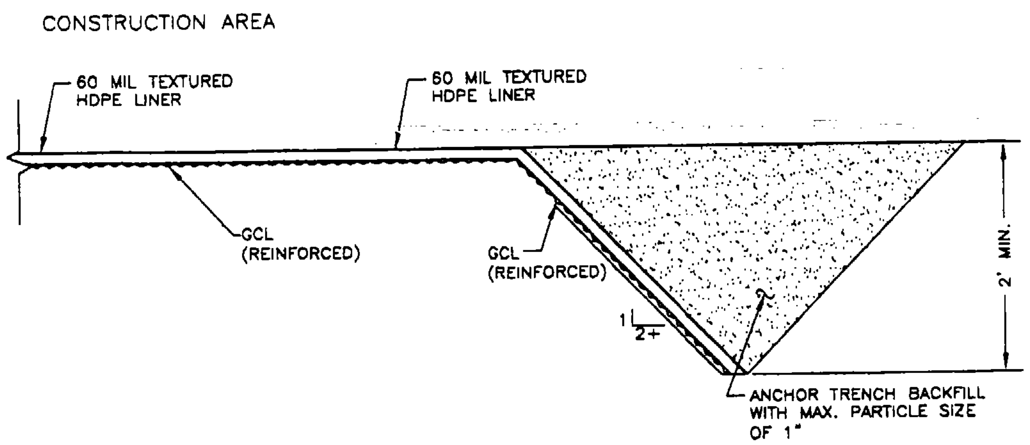
SCALE AS SHOWN

WASATCH REGIONAL

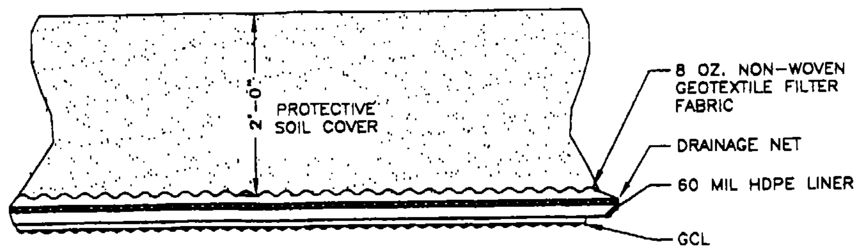
WASATCH REGIONAL LANDFILL FACILITY
LEACHATE WITHDRAWAL SYSTEM DETAILS

SHEET 9
113-30-100

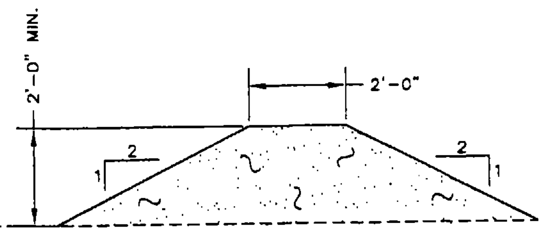
7/04



TYPICAL SIDE SLOPE GEOSYNTHETIC TIE-IN DETAIL 2/6
N.T.S.



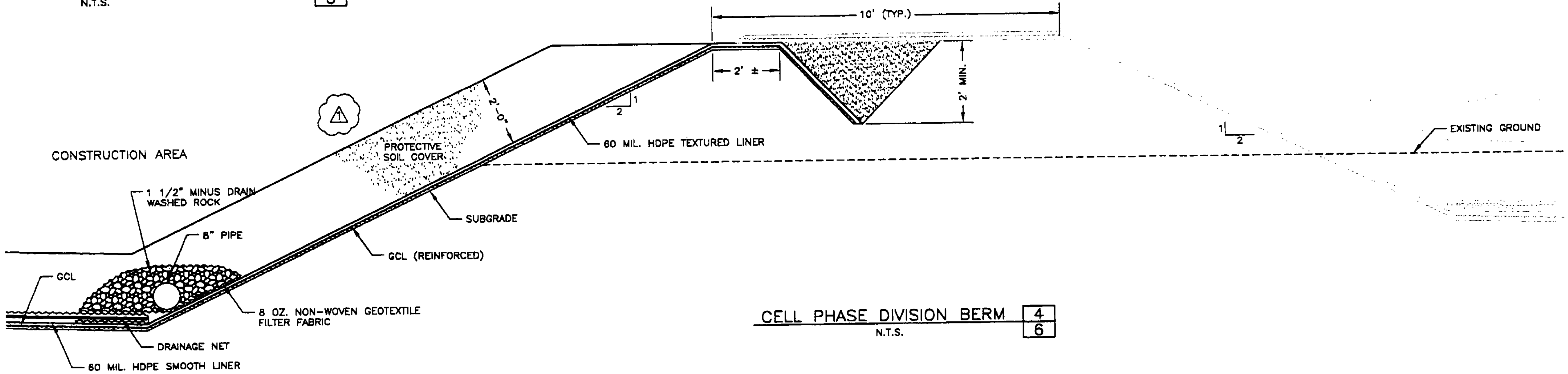
TYPICAL CELL FLOOR LINER SYSTEM SECTION B/–
N.T.S.



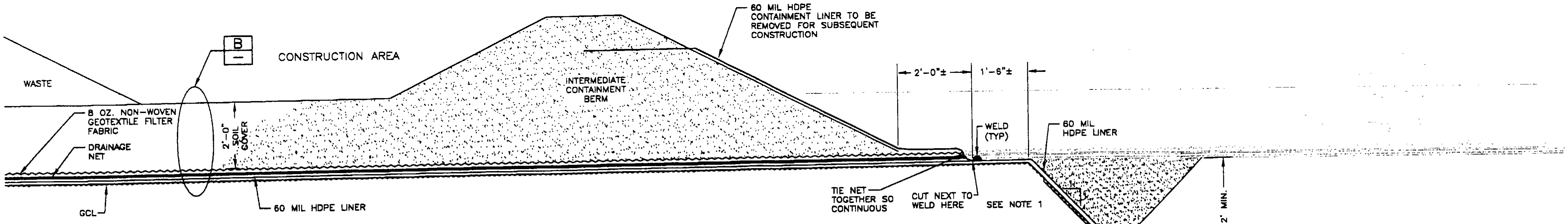
NOTES:

1. THE RUN-ON DIVERSION/SAFETY BERM IS TO BE CONSTRUCTED AROUND THE TOP PERIMETER OF CELL EXCAVATIONS WHERE NO RAISED EMBANKMENTS ARE CONSTRUCTED.
2. OWNER MAY DIRECT CONSTRUCTION OF A DITCH IN PLACE OF THE BERM.

TYPICAL RUN-ON DIVERSION/SAFETY BERM DETAIL 3/6
N.T.S.



CELL PHASE DIVISION BERM 4/6
N.T.S.



NOTE:

1. CUT ALONG WELD WITH LINER AND REMOVE 60 MIL. HDPE CONTAINMENT LINER TO ALLOW TIE-IN OF DRAINAGE NET, AND 8 OZ. NON-WOVEN GEOTEXTILE FILTER FABRIC.

TYPICAL INTERMEDIATE CONTAINMENT BERM SECTION 1/6
N.T.S.

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FILE DATE: 6.14.2005 08:28:08 (CAH)

HANSEN ALLEN & LUCE inc ENGINEERS

PROJECT ENGINEER

DESIGNED MPW, KCS	3				
DRAFTED JDB	2				
CHECKED KCS	1	06/2005	ADDED 2'-0" SOIL COVER TO CELL PAHSE DIVISION BERM	CAH	KCS
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY	APVD.

SCALE AS SHOWN

WASATCH REGIONAL

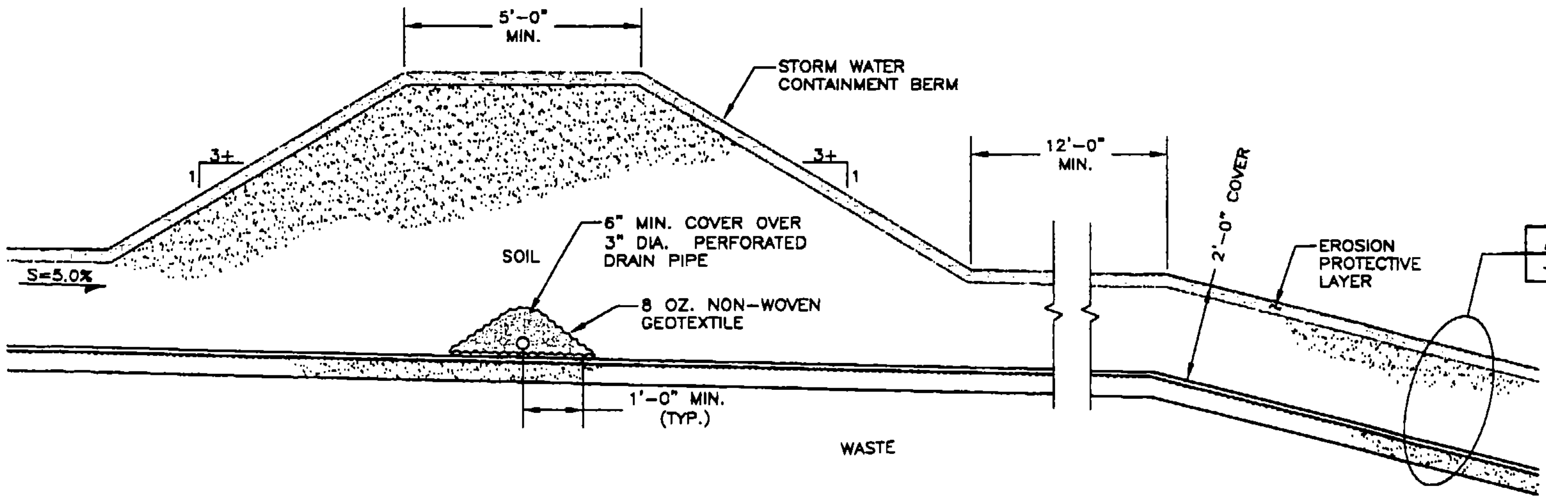
WASATCH REGIONAL LANDFILL FACILITY

TYPICAL LINER SYSTEM SECTIONS & DETAILS

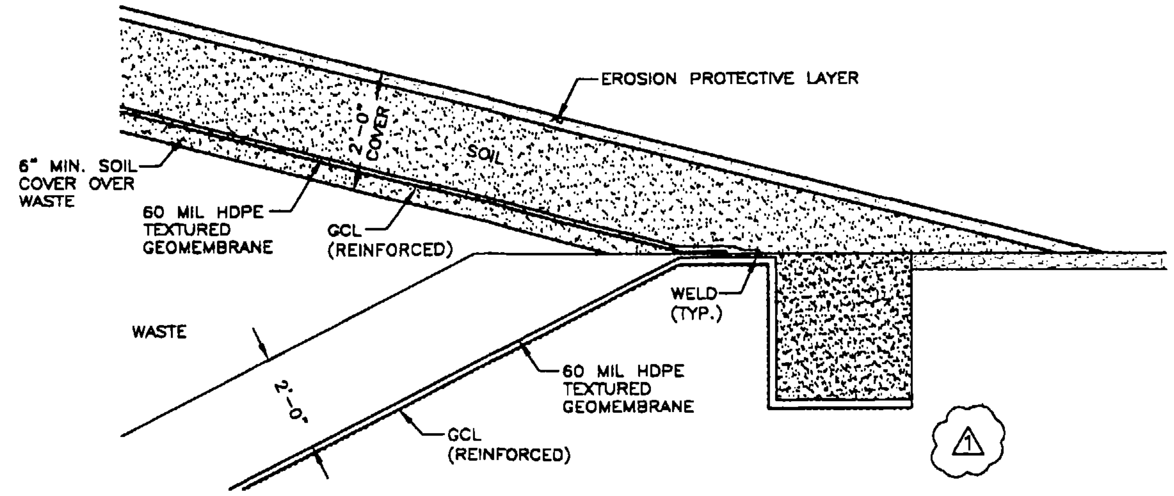
SHEET 10

113-30-100

7/04



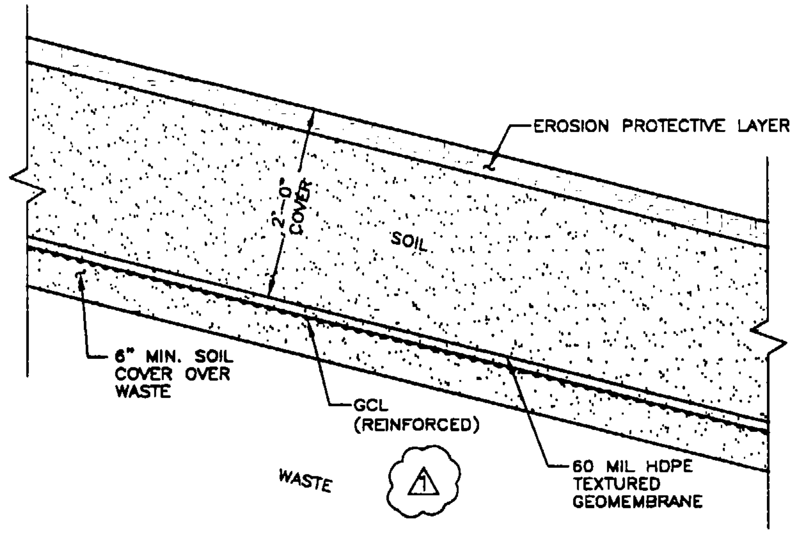
TOP OF 4:1 SLOPE SECTION 6/4
N.T.S.



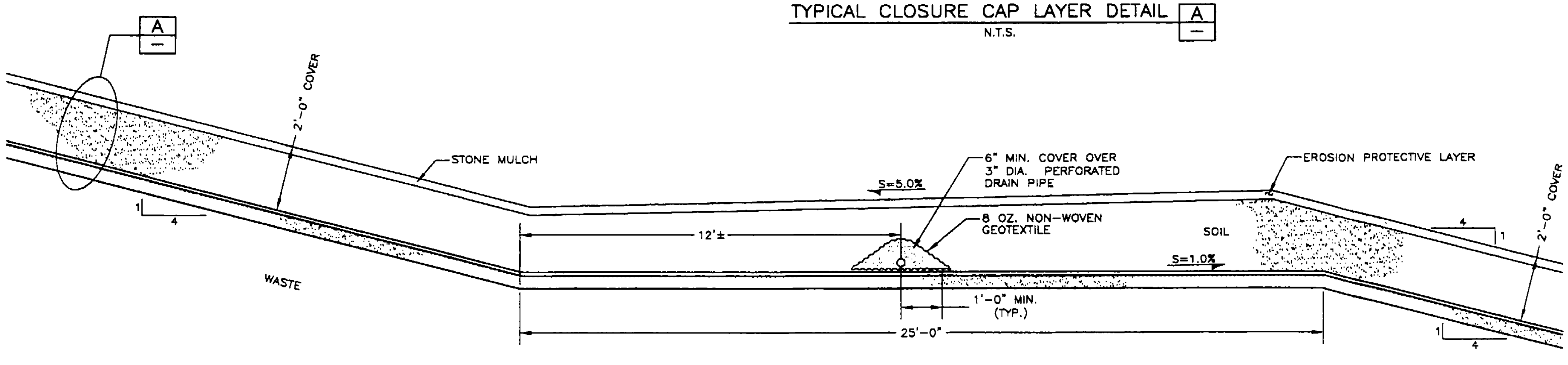
CAP TO CELL LINER TIE IN 8/4
N.T.S.

NOTES:

1. DRAIN PIPES UNDER STORM WATER CONTAINMENT BERMS AND UNDER BENCH DRAINAGE CHANNELS TO TIE INTO DOWN SPOUT INLET BOXES.
2. EROSION PROTECTION LAYER TO BE 3 INCHES THICK IF USING STONE MULCH OR 6 INCHES OF TOP SOIL WITH VEGETATION.



TYPICAL CLOSURE CAP LAYER DETAIL A/–
N.T.S.



TYPICAL CLOSURE CAP BENCH DRAINAGE CHANNEL SECTION 7/4
N.T.S.

DESIGNED MPW, KCS	3				
DRAFTED CAH	2				
CHECKED KCS	1	05/2005	ADDED CAP TO CELL LINER TIE IN DETAIL & GCL	CAH	KCS
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY	APVD.

HANSEN ALLEN & LUCE INC ENGINEERS

PROJECT ENGINEER

SCALE: AS SHOWN

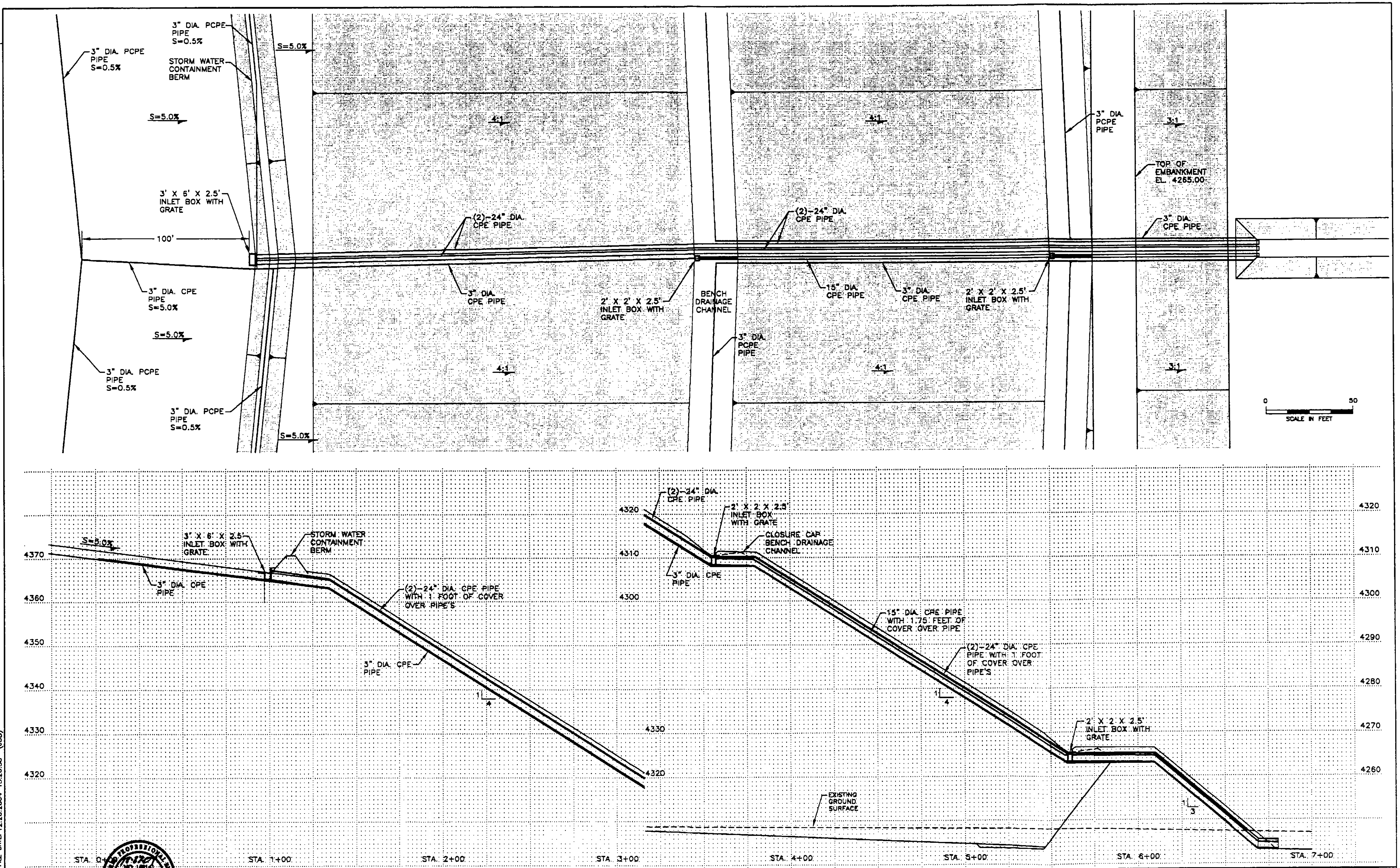
WASATCH REGIONAL

WASATCH REGIONAL LANDFILL FACILITY

CLOSURE CAP DETAILS

SHEET 11

113-30-100



HANSEN ALLEN & LUCE INC ENGINEERS

PROJECT ENGINEER

DESIGNED MPW, KCS	3			
DRAFTED DRB	2			
CHECKED KCS	1			
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY APVD.

SCALE AS SHOWN

WASATCH REGIONAL

WASATCH REGIONAL LANDFILL FACILITY

DOWNSPOUT PLAN & PROFILE

SHEET 12

113-30-100

FILE: ...113\30.100\CADFILES\PERMIT DWGS\DOWNSPOUT.DWG
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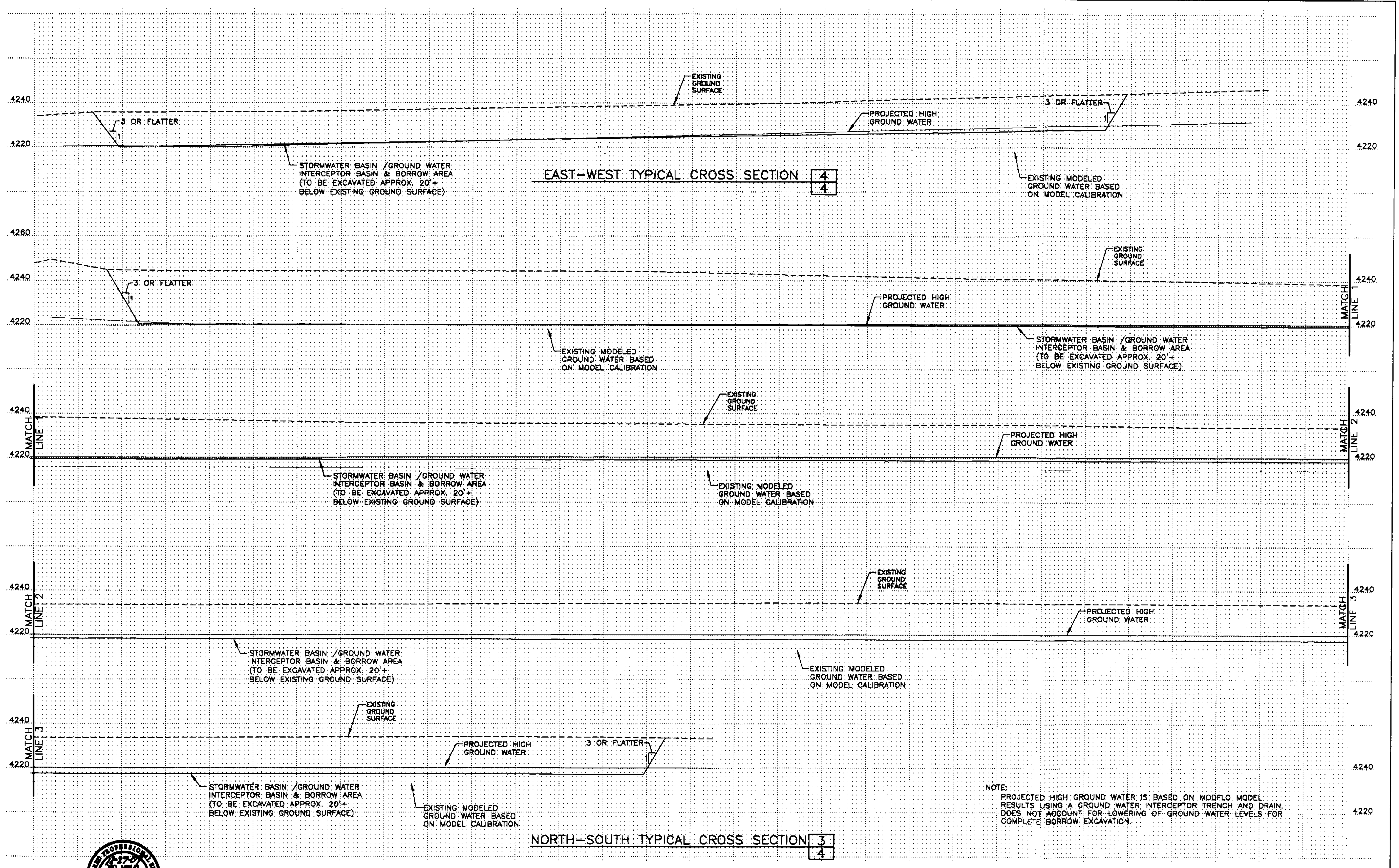

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HANSEN ALLEN & LUCE INC ENGINEERS

PROJECT ENGINEER

DESIGNED MPW, KCS	3				
DRAFTED DRB	2				
CHECKED KCS	1				
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY	APVD.

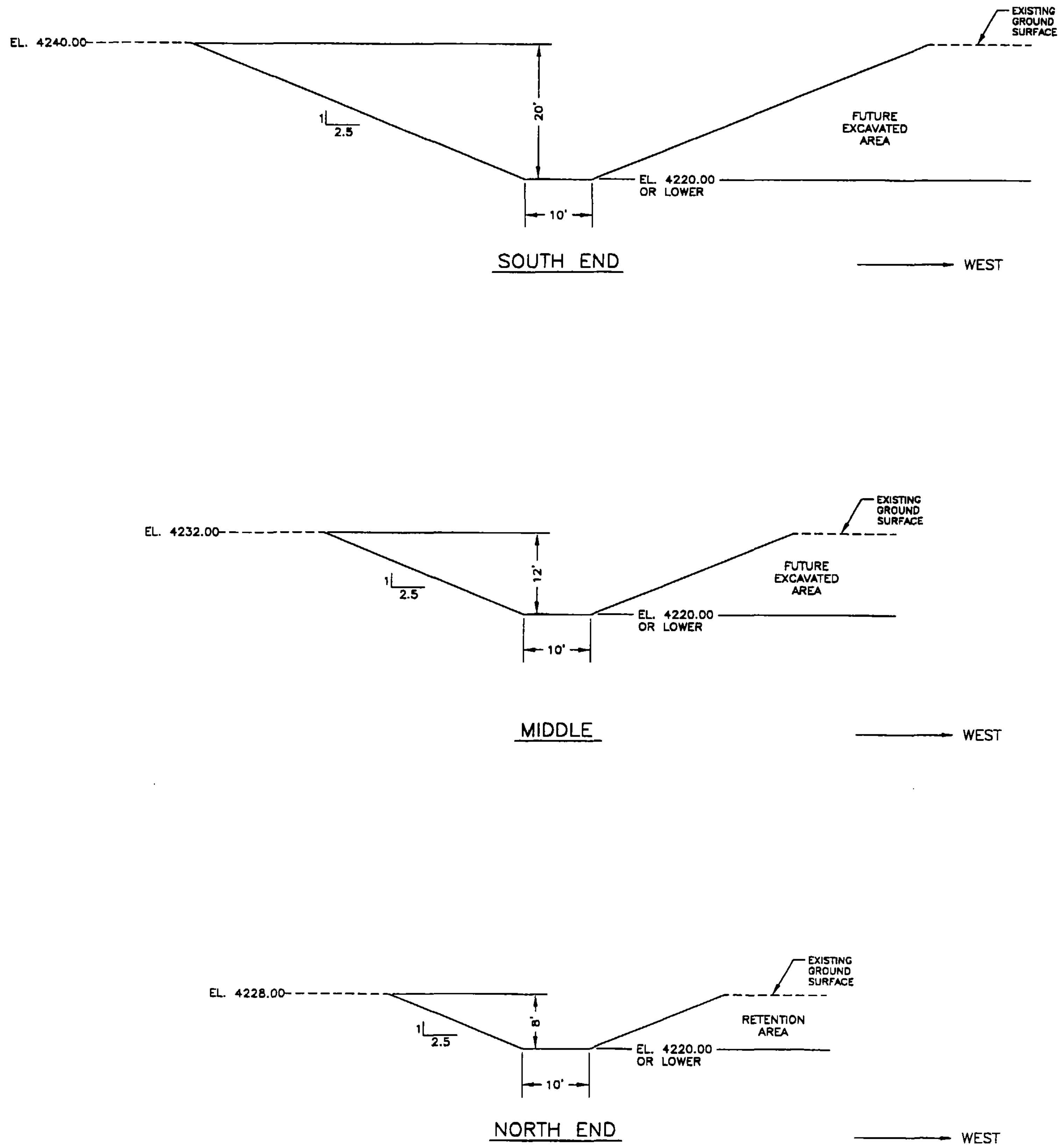
SCALE: HORIZONTAL 1" = 200', VERTICAL 1" = 40'

WASATCH REGIONAL

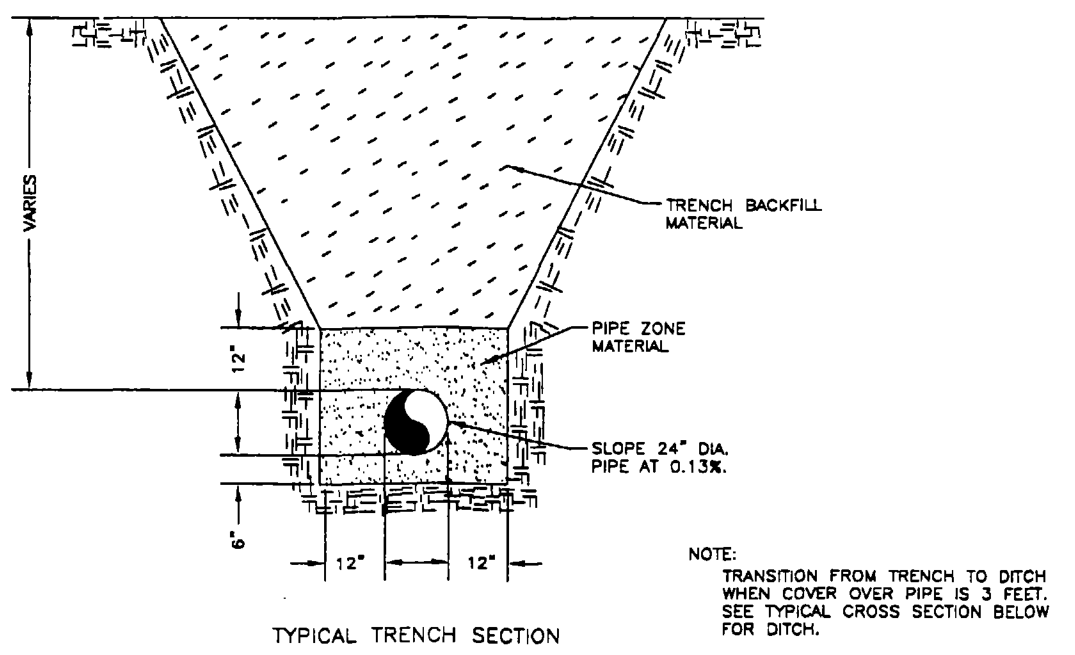
WASATCH REGIONAL LANDFILL FACILITY
GROUND WATER INTERCEPTOR &
STORM WATER BASIN SECTIONS

SHEET 13

113-30-100

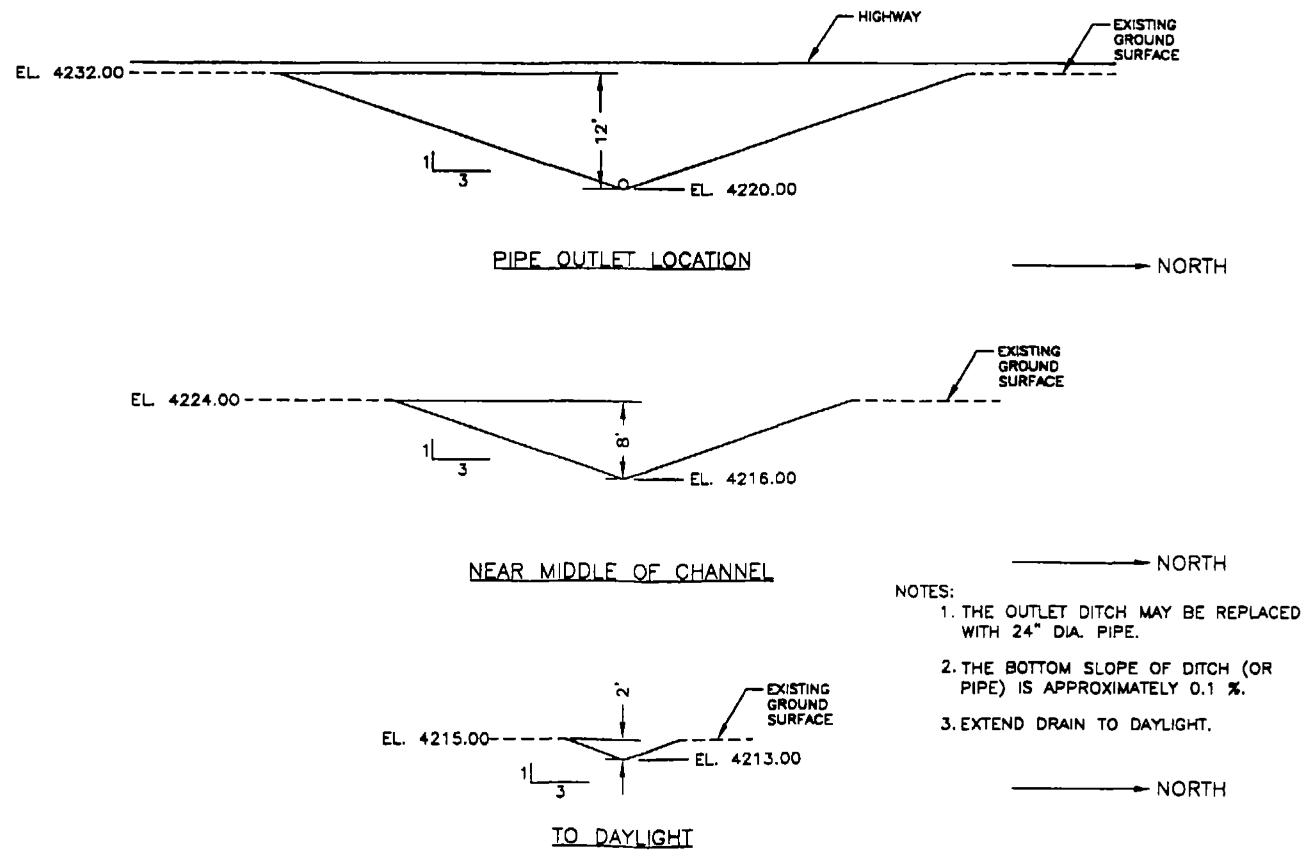


DRAIN TRENCH TYPICAL CROSS SECTIONS A/4



NOTE:
TRANSITION FROM TRENCH TO DITCH WHEN COVER OVER PIPE IS 3 FEET. SEE TYPICAL CROSS SECTION BELOW FOR DITCH.

DRAIN TRENCH OUTLET PIPE B/4



NOTES:
1. THE OUTLET DITCH MAY BE REPLACED WITH 24" DIA. PIPE.
2. THE BOTTOM SLOPE OF DITCH (OR PIPE) IS APPROXIMATELY 0.1 %.
3. EXTEND DRAIN TO DAYLIGHT.

DRAIN TRENCH OUTLET C/4

HANSEN ALLEN & LUCE INC ENGINEERS	PROJECT ENGINEER	DESIGNED MPW, KCS	3	
		DRAFTED CAH	2	
		CHECKED KCS	1	
		DATE DECEMBER 2004	NO.	DATE / REVISIONS / BY / APVD.

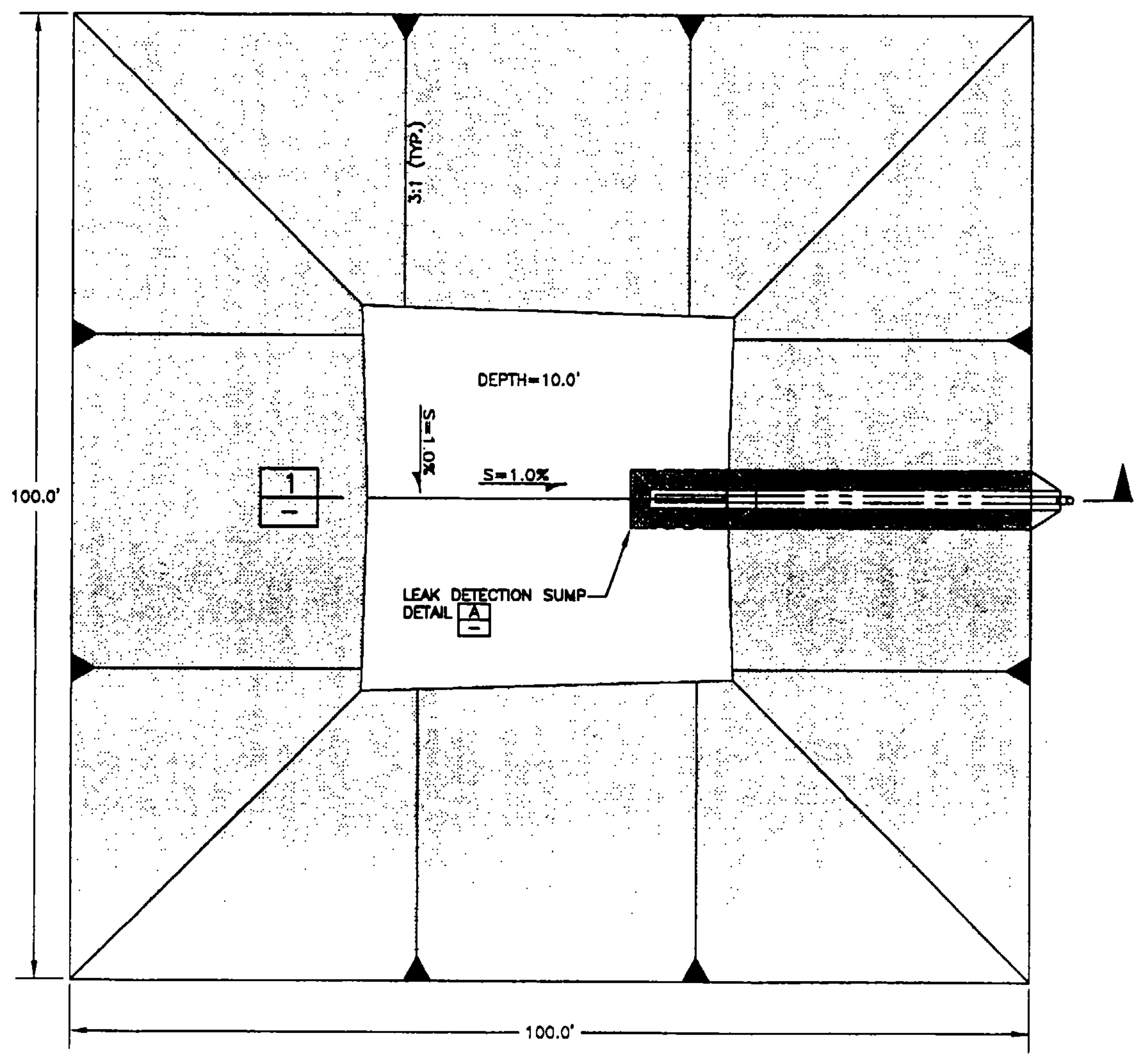
SCALE

WASATCH REGIONAL

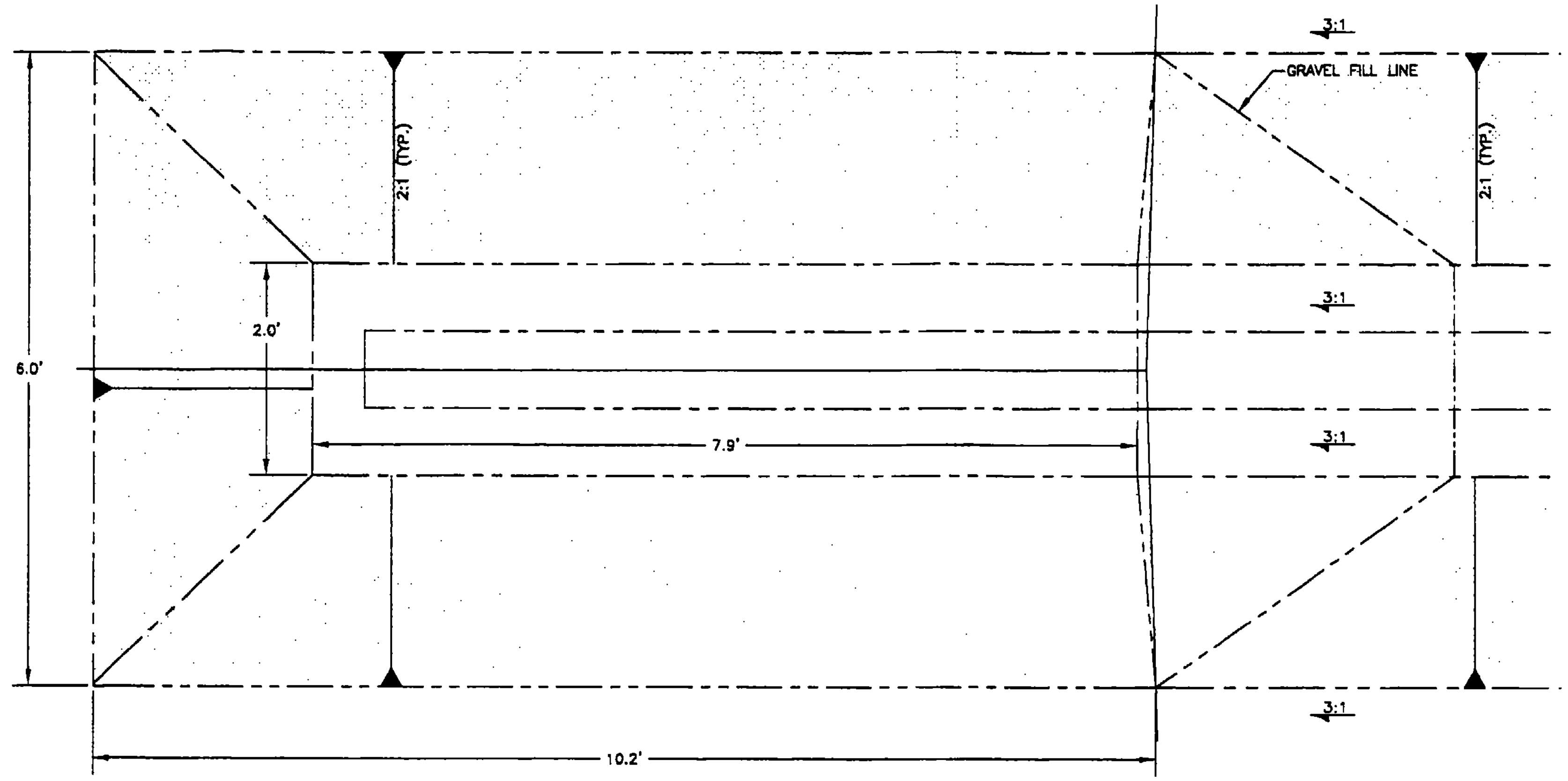
WASATCH REGIONAL LANDFILL FACILITY
GROUND WATER INTERCEPTOR &
STORM WATER BASIN OUTLET SECTIONS

SHEET 14

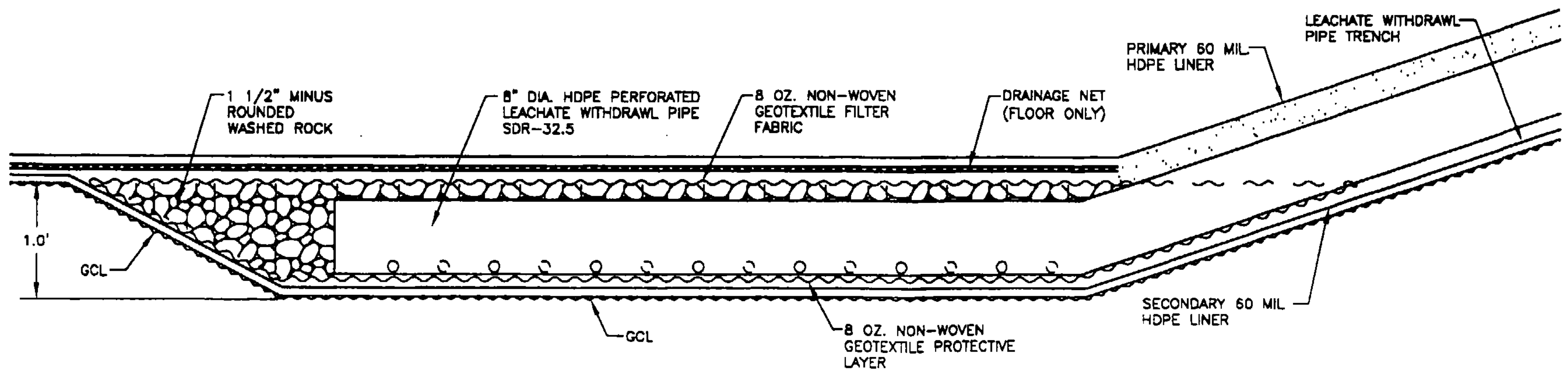
113-30-100



LEACHATE POND PLAN A/4
N.T.S.



LEAK DETECTION SUMP A/–
N.T.S.



LEAK DETECTION SUMP SECTION 1/–
N.T.S.

HANSEN ALLEN & LUCE INC ENGINEERS	PROJECT ENGINEER	DESIGNED MPW, KCS	3			
		DRAFTED	2			
		CHECKED KCS	1			
		DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY / APVD.

SCALE

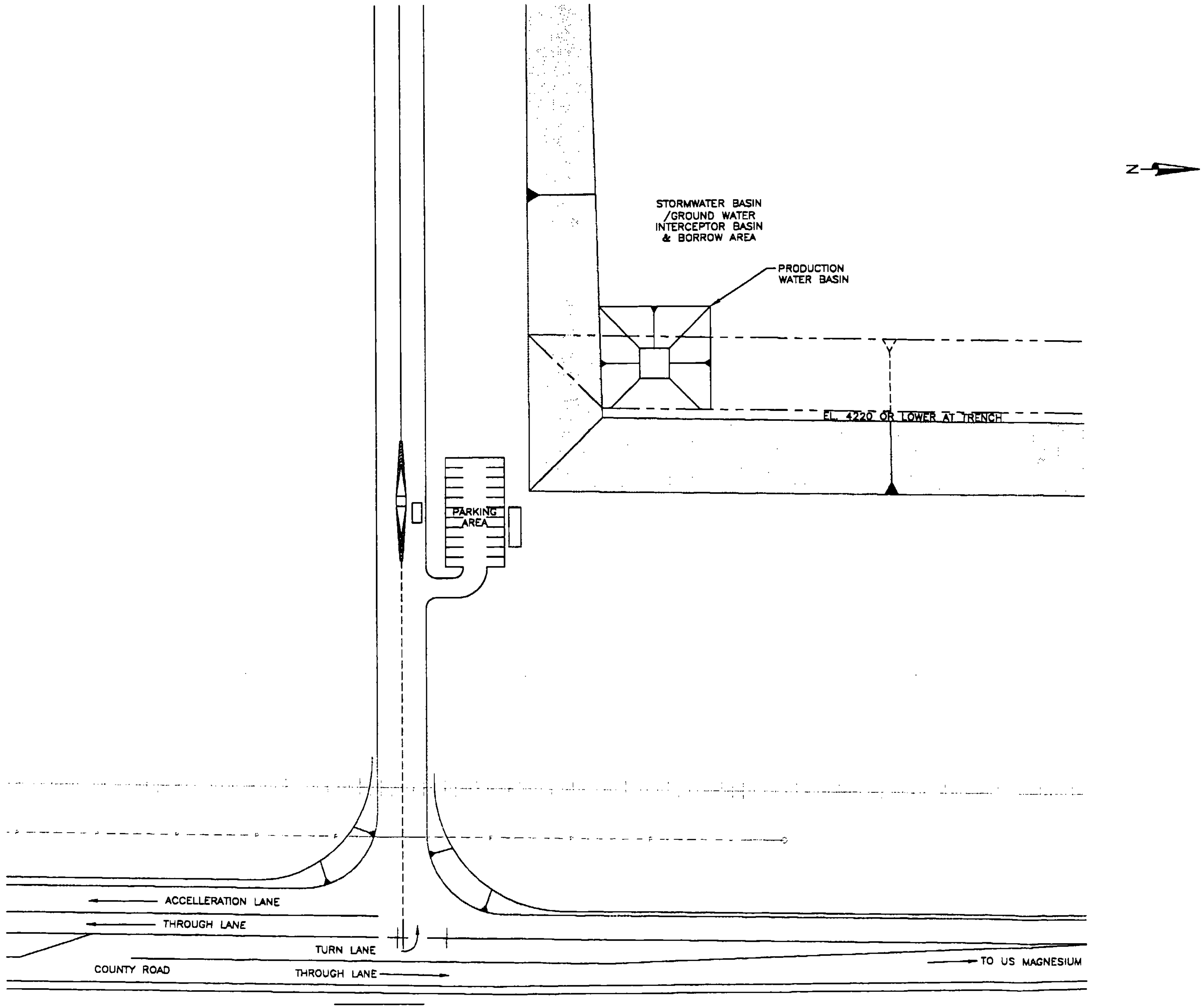
WASATCH REGIONAL

WASATCH REGIONAL LANDFILL FACILITY

LEACHATE EVAPORATION POND DETAILS

SHEET 15

113-30-100



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FILE DATE: 12.20.2004 13:24:27 (JOB)

HANSEN ALLEN & LUCE INC ENGINEERS

DESIGNED MPW, KCS	3					
DRAFTED CAH	2					
CHECKED KCS	1					
DATE DECEMBER 2004	NO.	DATE	REVISIONS	BY	APVD.	

PROJECT ENGINEER

SCALE: NOT TO SCALE

WASATCH REGIONAL

WASATCH REGIONAL LANDFILL FACILITY

FACILITY ACCESS ROAD

SHEET 16

113-30-100

7/04

APPENDIX B

GEOTECHNICAL INVESTIGATION PERMIT MODIFICATION

WASATCH REGIONAL SOLID WASTE LANDFILL

PREPARED BY
APPLIED GEOTECHNICAL
ENGINEERING CONSULTANTS

DECEMBER 17, 2004

Revised June 15, 2005

Applied Geotechnical Engineering Consultants, P.C.

HAND DELIVERED
05.02126
JUN 17 2005
UTAH DIVISION OF
SOLID & HAZARDOUS WASTE

GEOTECHNICAL INVESTIGATION
PERMIT MODIFICATION

WASATCH REGIONAL SOLID WASTE LANDFILL
SECTION 33 AND WEST HALF SECTION 34
TOWNSHIP 2 NORTH, RANGE 8 WEST
AND SECTION 4, WEST HALF SECTION 3
TOWNSHIP 1 NORTH, RANGE 8 WEST
TOOELE COUNTY, UTAH

PREPARED FOR:

WASATCH REGIONAL LANDFILL
C/O HANSEN, ALLEN AND LUCE INCORPORATED
6771 SOUTH 900 EAST
MIDVALE, UTAH 84047

ATTENTION: KENT STAHELI

PROJECT NO. 1040644

DECEMBER 17, 2004
REVISED JUNE 15, 2005

600 West Sandy Parkway • Sandy, Utah 84070 • (801) 566-6399 • FAX (801) 566-6493

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LABORATORY TEST RESULTS . . . . Page 9

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FREE WATER . . . . Page 14

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- D. Liquefaction . . . . Page 23

GCL COMPATIBILITY . . . . Page 24

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EXECUTIVE SUMMARY

1. The natural soil and bedrock at the site are suitable for support of the proposed landfill disposal facility.

2. Exterior slopes of 3:1 and interior cut and fill slopes of 2:1 (horizontal to vertical) may be used for the base of the landfill facility.

   The final exterior slope of 4:1 will provide satisfactory stability of the waste pile.

4. The natural soil is suitable to use in construction of the proposed embankment.

5. As proposed, a geosynthetic clay liner will also provide appropriate stability along with the other synthetic materials for the interior landfill bottom and also the closure cap.

6. Bentonite from a GCL was tested with water leached from soil samples at the site indicate a permeability of 1.5×10^{-9} cm/sec.

7. Design details and construction precautions are contained in the text of the report.

SCOPE

This report presents the results of a geotechnical investigation for the permit application of the proposed Wasatch Regional Solid Waste Landfill. The facility is to be located west of Rowley Road, approximately 6 miles north of Interstate 80 within the western half of Section 3 and Section 4 of Township 1 North, of Range 8 West along with the western half of Section 34 and Section 33 of Township 2 North, Range 8 West, Salt Lake Base and Meridian in Tooele County, Utah. The revision to the report was requested to include a geosynthetic clay liner (GCL) between the flexible membrane liner (FML) and the cover material on the closure cap.

The subsurface information, geology, seismic conditions along with characteristics of the on-site materials contained within a geotechnical report for the Wasatch Regional Solid Waste Landfill in Tooele County, Utah prepared by Kleinfelder and reported on May 18, 2004 under their File No. 35467.003 has been relied upon in this study.

This report provides the information requested in our proposal dated July 15, 2004 addressed to Allied Waste in care of Hansen, Allen and Luce Incorporated. The items requested for this study include the following:

- Characterize the subsoils.
- Determine the suitability of the subsoils for support of the proposed landfill.
- Provide recommendations for foundation preparation for the landfill.
- Provide recommendations for embankments that would be constructed in conjunction with the landfill.
- Stability issues using geosynthetics as liner and drainage materials.
- Compatibility of the GCL with the on-site soil and water.
- Seismic characteristics.
- Stability analysis of the closed facility.
- Stability analysis during waste placement.
- Suitability of the on-site soil for use as fill.

- Recommendations for imported fill.
- Fill material compaction criteria.

PROPOSED CONSTRUCTION

We understand that the proposed landfill will be developed by placing an embankment on the east portion of the facility close to the existing elevation of 4246 to 4240 feet. At that point, an embankment would be constructed with a slope of approximately 3:1 (horizontal to vertical) extending up to an embankment crest elevation of 4265. A 25 foot horizontal bench would then be provided with the interior portion of the embankment sloping down into the landfill area at a 2:1 (horizontal to vertical) slope to an elevation of approximately 4244 feet. The floor of the landfill would then extend west at a slope of 1.7 and 1.2 percent. At the end of the floor, the ground surface would then slope up at a 2:1 (horizontal to vertical) slope to the west edge of the landfill. This 2:1 slope will be cut and when needed will receive soil as fill to protect the overlying geosynthetics.

The interior surface of the landfill will be prepared to receive waste by having the following materials placed on the floor, from top down.

Two feet of protective soil cover
Non-woven geotextile
Drainage net
Flexible membrane liner (HDPE)
Geosynthetic clay liner (GCL)
Prepared Subgrade

On the 2:1 interior side slopes, the profile would consist of from top down:

Two feet of protective soil cover (as far up the slope to limit stress on the liner materials)

Flexible membrane liner (HDPE textured)

Geosynthetic Clay Liner (GCL)

Prepared Subgrade

The final configuration of the landfill will extend approximately 100 feet vertical feet from the west inside edge of the embankment up at a 4:1 slope. Included with the slope will be two horizontal benches approximately 25 feet wide. At the top of the 4:1 slope, a small berm will be placed in order to prevent drainage from extending down the slope. The top of the landfill will slope up towards the west at an approximate 5 percent slope. The west edge of the cap will slope down at a 4:1 slope to natural soil.

The profile of the materials on the closure cap will consist of the following (from top down):

Two foot cover material including soil and an erosion protective layer

Textured Flexible Membrane Liner (HDPE)

Geosynthetic Clay Liner (GCL)

Protective soil (approximately 6 inches)

Waste

The 4:1 side slopes will have the following profile (from top down):

Two foot cover material including soil and an erosion protective layer

Textured Flexible Membrane Liner (HDPE)

Geosynthetic Clay Liner (GCL)

Protective soil (approximately 6 inches)

Waste

We anticipate that waste placement will begin at the eastern end (the lowest elevation) and proceed in horizontal lifts until the final profile is achieved.

Approximately 300 feet east of the toe embankment will be the beginning of a borrow area for construction and daily cover soil. It is anticipated that the natural soils will be excavated down to a depth of approximately 20 feet with a perimeter slope of approximately 3:1 and flatter. This area of excavation will extend to within approximately 300 feet of the railroad tracks that parallel Rowley Road.

SITE CONDITIONS

The site is currently vacant of permanent structures with a few dirt roads on the property. The ground surface within the area of the proposed facility currently slopes down towards the east at a slope of approximately 5 percent. Near the toe of the proposed facility, the ground surface is fairly flat.

The site is basically at the foothill of the Lakeside Mountains. Further to the east, the ground surface slopes down to the Great Salt Lake. The lake at its current location is approximately 5 to 6 miles to the east/northeast.

FIELD INVESTIGATION

The subsurface conditions for this phase of the study was conducted by drilling five borings at the locations indicated on Figure 1. Three of the borings were advanced to ground water and monitoring wells constructed. The drilling extended down to a maximum depth of 173 feet. Drilling was initially started using 8-inch, hollow-stem auger powered by an all-terrain (CME 750) drill rig. For the deeper exploration and in more difficult drilling conditions, rotary methods using a 3½ inch diameter tricone bit was used with air as the circulation fluid.

Samples were obtained, with a California spoon sampler with an automatic hammer advancing the samplers. Disturbed bulk samples were also obtained from the cuttings.

The holes constructed to be monitoring wells were completed by estimating the water level and then placing a 15 to 20 foot section of screen with openings of 0.010 inches. A 5 foot section of PVC pipe was placed below the screened portion and solid pipe extended above the screen portion up to the ground surface. Sand was placed within the annular space within the screened section (and 1 to 8 feet above the screened portion) with bentonite chips being used to backfill from the sand portion up to near the ground surface. Concrete was placed in the upper 1¼ feet. The soil borings were backfilled with cuttings.

The California sampler (2 inch diameter) was advanced by driving with blows from a 140 pound automatic hammer falling 30 inches. This test is similar to the standard penetration test as described by ASTM Method D-1587, except the sampler used is a 2 inch diameter sampler as opposed to a 1⅜ inch inside diameter sampler.

Based on studies conducted by Goodman and Carol (Goodman and Carol, Theory and Practice of Foundation Engineering, the McMillam Company, New York, 1968, p 54), the actual measured penetration resistant values obtained using the California sampler should be multiplied by 0.82 to equate them with the penetration resistant values using the standard penetration sampler. Penetration resistant values, when properly evaluated, provide an indication of relative density or consistency of the soils encountered.

Measurements were made in the borings to determine the presence of free water. Water measurements obtained after completion of exploratory borings are shown on the logs of exploratory borings.

LABORATORY TESTING

Laboratory testing was conducted on selected samples of the natural soils in order to determine their engineering characteristics. Laboratory testing conducted during the study includes: natural moisture content, dry density, Atterberg Limits, grain-size distribution, strength, moisture/density relationship and consolidation. The test results are shown on Figures 6 through 18. A summary of the laboratory test results is shown on Table I.

A discussion of laboratory testing procedures is presented below. The testing procedures are primarily those of American Society for Testing and Materials (ASTM).

Index Properties - The unified soil classification system (ASTM D-2487) was used to classify the soil. This system is based on index property tests including the determination of natural water content (ASTM D-2216), liquid and plastic limits (ASTM D-4318) and grain-size distribution (ASTM D-422). Results of the moisture content, dry density, Atterberg Limits and percentage of soil passing the No. 200 sieve are presented on Table I.

Consolidation - Consolidation tests were performed during this investigation. Consolidation test samples were prepared and placed in a consolidometer ring between porous disks. An initial seating load of 500 pounds per square foot was placed on the sample. The sample was then loaded to 1,000 pounds per square foot. The percent change in sample heights was measured with a dial gauge as the sample was wetted and loaded incrementally until a straight line relationship between load and strain was obtained. In two cases, the loads were reduced to measure the rebound portion of the consolidation curve. The consolidation test procedure described is similar to ASTM Method D-2435. Results of consolidation tests are plotted as a curve of the final strain at each increment of pressure against the log of accumulated pressure. These tests are shown on Figures 12 through 14.

Triaxial Shear - A triaxial shear test was performed in general accordance with ASTM D-4767. The sample was prepared by trimming the ends perpendicular to the sample axis and placing it in a latex membrane. The prepared sample was placed in the triaxial cell and was saturated using back pressure saturation. Testing continued by placing a consolidation load of 7 psi and then shearing the sample to near failure. The sample was then reconsolidated at 14 psi and then again sheared to near failure. The sample was then consolidated at 28 psi and this time sheared to failure. Sample strains, loads and pore pressures were monitored throughout each stage of the test. The test results are shown on Figure 8.

Direct Shear - Direct shear tests were conducted in general accordance with ASTM D-3080 on undisturbed samples of the soil. Each sample was consolidated at loads of 1, 2 and 4 kips per square foot. After each of the consolidation pressures, the sample was sheared with the peak strength being obtained. The test results are presented on Figures 9, 10 and 11.

Leached Water - Four samples of on-site soil were returned to the laboratory and were used to obtain water leached from the soil. This process was conducted in accordance with ASTM D-6151. The leached water was then used to measure the Atterberg Limits of two possible sources of bentonite for the geosynthetic clay liner, and also was used as the permeant in a permeability test of a GCL bentonite.

Permeability - Bentonite taken from a sample of the potential geosynthetic clay liner was tested for permeability using one of the leachates obtained from the on-site soil. The test was conducted following ASTM D-5084-90 procedure.

LABORATORY TEST RESULTS

Listed below is a summary of the index properties for the soils encountered by AGEC and also Kleinfelder.

Soil Index Properties

Soil Type	Gravel (percent)	Sand (percent)	Clay Silt (percent)	Liquid Limit (percent)	Plasticity Index (percent)
Lean Clay	0 - 1 (0)	10 - 33 (25)	51 - 97 (28)	26 - 102 (44)	10 - 53 (18)
Silty Clay	0 - 1 (0)	21 - 36 (28)	51 - 87 (71)	21 - 49 (30)	0 - 19 (9)
Silty Sand	0 - 20 (7)	49 - 92 (73)	5 - 66 (31)	20 - 29 (22)	0 - 9 (2)
Sandy Gravel	11 - 70 (47)	20 - 35 (30)	8 - 56 (29)	40	26

Note: The values above are the ranges of samples tested within the general deposit. The numbers in ( ) are average values.

The engineering characteristics of the natural soils were also determined by the consolidation and strength tests. Listed below is a summary of the strength and compressibility characteristics.

Strength - Direct Shear Test

Location	Tested by	Friction (degrees)	Cohesion (psf)	Remarks
B - 2 @ 2'	Kleinfelder	35	550	Remolded to 95%
B- b @15'	Kleinfelder	29	75	Remolded to in-situ conditions
B - 10 @ 10'	Kleinfelder	31	0	Remolded to in-situ conditions
B - 2 @ 34'	AGEC	35	40	Undisturbed
B - 3 @ 14'	AGEC	33	0	Undisturbed
B - 4 @ 14'	AGEC	30	100	Undisturbed

Strength - Triaxial Shear Test

Location	Tested by	Friction (degrees)	Cohesion (psf)	Remarks
B - 4 @ 24'	AGEC	32	80	Effective Stress Parameters
		26	160	Total Stress Parameters

Strength - Unconfined Compression Test

Location	Tested by	Compressive Strength (psf)
B - 11 @ 10'	Kleinfelder	3580

Consolidation Testing

Boring	Depth	Tested by	Cr′	Cc′	mpp	Description
B - 2	5'	Kleinfelder	0.018	0.177	900	Lean Clay w/Sand
B - 3	7½′	Kleinfelder	0.014	0.005	7000	Sandy Lean Clay
B - 4	15'	Kleinfelder	0.022	0.064	2000	Sandy Lean Clay
B - 5	7½′	Kleinfelder	0.007	0.108	5000	Sandy Silty Clay
B - 9	8'	Kleinfelder	0.015	0.081	4000	Clayey Sand
B - 9	30'	Kleinfelder	0.022	0.118	4200	Elastic Silt
B - 11	10'	Kleinfelder	0.010	0.165	2200	Silt w/Sand
B - 1	68'	AGEC	0.01	0.092	—	Sandy Lean Clay
B - 3	29'	AGEC	0.008	0.101	2000	Lean Clay
B - 4	19'	AGEC	—	0.070	—	Sandy Silt

In order to determine the potential impact of dissolvable salts on the performance of bentonite from the GCL, leached water from four soil samples at the site and were used to conduct Atterberg Limit tests and a permeability test. The test results from the soil samples and the effect of the leached water on the Atterberg Limits are listed below:

Location of Leached Soil Sample

Sample Designation	Sample Location
A	Northwest Area of Property
B	Midpoint on South Side of Property
C	Near Kleinfelder B-3
D	Near Kleinfelder B-5

The index properties of the soils tested of the samples obtained are indicated below:

Leached Soil Index Properties

		Gradation			Atterberg Limits	
Sample	Moisture Content (%)	Gravel +4 (%)	Sand -4 & +200 (%)	Silt/Clay 200 (%)	Liquid Limit (%)	Plasticity Index (%)
A	6	1	60	39	22	6
B	6	0	9	91	18	1
C	5	0	18	82	22	6
C	2	0	61	39	17	2

Listed below is a summary of the test results using this water with the two different bentonites.

Atterberg Limits with Various Water Sources

Water Source	Atterberg Limit Test Results			
	Cetco bentonite		GSE bentonite	
	LL	PI	LL	PI
Distilled Water	492	470	532	503
Site Piezometer Water	353	329	284	255
Sample A Leached Water	306	281	264	240
Sample B Leached Water	461	437	524	492
Sample C Leached Water	411	387	439	409
Sample D Leached Water	352	328	289	256

The permeability of the GSE bentonite using Sample A leached water was measured to be 1.5×10^{-9} cm/sec.

SUBSURFACE CONDITIONS

Subsurface conditions at the site were characterized by the exploratory borings drilled by AGEC and the subsurface information reported by Kleinfelder. The subsurface profile consists of clay, silt and fine sand on the lower elevation portions of the site with more granular materials being encountered on the higher elevation portions of the site. Bedrock was encountered in one of the borings at a depth of 143 feet (Boring B-1). The bedrock was found to be limestone.

A general description of each of the soil types encountered in the borings is indicated below:

<u>Lean Clay</u> - The lean clay was found to be interlayered with sandy silt and occasionally some silty sand. The clay was found to be stiff to very stiff, slightly moist to moist and brownish gray in color.

Silty Clay - The silty clay was found to be sandy and medium to soft and wet. The color of was found to be gray.

Silty Sand - The silty sand was found to contain occasional lean clay layers. The silty sand was found to be loose to dense. The moisture condition varied from moist to wet and the color was gray to grayish brown.

Sandy Gravel - The sandy gravel was found to be silty and clayey. Occasional cobble and boulders were also encountered. The density of this deposit was found to be medium to very dense. The moisture condition was generally moist to wet and the color was brownish gray.

Bedrock - The bedrock encountered consisted of limestone. It was also found to be gray.

FREE WATER

Water was encountered in the deeper borings at an approximate elevation of 4220 to 4235.

EMBANKMENT

A. Section

A typical embankment section for the proposed landfill cell is shown on Figure 19. The proposed section as described earlier, consists of an exterior slope of 3:1 and an interior slope of 2:1 (horizontal to vertical). The embankment will have a top crest width of 25 feet at a top elevation of 4265. It is our understanding that the embankment will be constructed as a homogeneous compacted earth fill section with synthetic materials on the interior portion of the slope. The overall exterior height will be from 15 to 19 feet. With the top elevation of 4265 and the interior toe elevation of 4244, the interior 2:1 slope will be 21 feet high.

B. Stability

Stability of the proposed embankment and landfill was analyzed under several loading conditions. Factors of safety for the embankment were determined with respect to mass rotational and sliding wedge failures. Static and dynamic (pseudo) static analysis of the embankment was conducted using the configuration discussed above.

1. Soil Profile

The soil profile used in the stability analysis of the embankment and landfill was defined from the information obtained from the exploratory borings and laboratory test results. The soil profile assumed is the weaker of the materials encountered and consists of clay, silty clay and silty sand. A graphic presentation of the soil profile used in the analysis is shown on Figure 19.

2. Moisture Conditions

No free water was included in the evaluation of the embankment slope other than the ground water elevation of 4235 feet was on the east and up to 4260 on the west.

The potential of water entering the embankment would be limited to surface infiltration from the exterior portion of the embankment. The interior portion of the embankment will be covered with impervious synthetic liners. With this condition, the embankment and foundation soils were evaluated assuming drained conditions. Due to the significant amount of sand, the interlayered conditions of the fine-grained soil and the extended period of time for placement of fill and waste, the natural soils were evaluated under drained conditions.

3. Seismic Considerations

The seismic conditions, as reported by the USGS (2003) were used to evaluate the stability of the embankment under seismic conditions. The USGS indicates an acceleration that has a 2 percent probability of exceedance in 50 years (10 percent in 250 years) results in an acceleration of approximately 0.210g.

This acceleration was adjusted for the stability analysis as recommended in the DMG Special Publication 117 "Guidelines for Analyzing and Mitigating Landslide Hazards in California". Using this document, an acceleration of 0.092g was used for the stability calculations assuming a threshold 15cm displacement.

4. Strength Parameters

The strength parameters used for the stability analysis were determined from the field and laboratory test results conducted in this study and also by Kleinfelder. The testing consisted of penetration resistances, unconfined compressive strength tests, triaxial shear tests and direct shear tests conducted on undisturbed and remolded soil samples. Based on these results, previous testing by others and our judgment, strength parameters for each material were selected.

A table summarizing the waste and soil materials and their strengths is indicated below:

Strength Parameters - 1

Material	Unit Weight (pcf)	Friction (degrees)	Cohesion (pcf)
Waste	120	25	100
Embankment	120	32	300
Clay, Silt, Silty Sand (Fine)	105	31	40
Gravel (Coarse)	130	37	0

A table summarizing the synthetic/soil materials and their internal and interface strength parameters are listed below:

Strength Parameters - 2

	Internal		Interface	
	Friction (degrees)	Cohesion (psf)	Friction (degrees)	Cohesion (psf)
A - Floor				
Waste	25	100		
			25	100
Soil Cover	25	100		
			21	80
Non-woven Geotextile	—	—		
			8	0
Drainage Net	—	—		
			9.4	0
HDPE	—	—		
			8	0
GCL	18	50		
			26.8	30
Soil	31	40		
B - Side Slope (2:1 Slope)				
Waste	25	100		
			25	100
Soil Cover	25	100		

	Internal		Interface	
	Friction (degrees)	Cohesion (psf)	Friction (degrees)	Cohesion (psf)
			23.9	95
HDPE (Textured)	—	—		
			21	250
GCL	18	50		
			26	30
Soil	31	40		
C - Cap (4:1 Slope)				
Soil	25	100		
			23.9	95
HDPE (textured)	—	—		
			21	250
GCL	18	50		
			21	80
Soil	25	100		
			25	100
Waste	25	100		
D - Cap (top)				
Soil	25	100		
			21.4	84
HDPE (textured)	—	—		
			21	260
GCL	18	50		
			21.4	8.4
Soil	25	100		
			25	100
Waste	25	100		

The interface strength parameters where specific test values were not available were selected by taking the weaker strength of 1) the adjacent material, 2) approximately 84 percent of the weaker materials if a smooth synthetic material is included or 3) 95 percent of the weaker materials if a textured synthetic is included.

5. End of Construction - Long Term Conditions

Typically, in a clay soil environment, construction of an embankment may induce excessive pore pressure in the foundation soil. With the excessive pore pressure, the friction resistance of the clay soils against sliding may not increase with the addition of load. To model this condition where the excess pore pressures reflect the addition of embankment material or waste, an end of construction analysis is conducted of the embankments.

Under long term conditions, excess pore pressures which may have developed during construction are assumed to have dissipated, thus mobilizing the friction resistance available in the foundation soils. We have assumed this condition under the long-term condition and during placement of waste within the landfill. We anticipate that the landfill is large enough and that the placement of waste would not result in a significant increase of pore pressure.

With the clay, silty sand to sandy silt material used for embankment construction, the strength parameters for both end of construction and long term conditions for the embankment were assumed to be in a drained condition.

6. Bearing Capacity

Soil bearing capacity with respect to the proposed landfill was evaluated. The stability calculations summarized in the next section also models a bearing capacity type failure. A bearing capacity type failure is defined as the lack of strength within the foundation soils versus support of the proposed construction. Typically, the bearing capacity of an embankment is evaluated by conducting stability analysis.

Classical bearing capacity calculations have been conducted to determine the bearing capacity of the natural soils with respect to the proposed embankment construction and under the loading conditions resulting from completed disposal cell. A safety factor greater than 3 with regards to classical bearing capacity is calculated for the embankment alone at the level of the softest natural soils. In these calculations, it was assumed that the soft clay material extends to great depth.

Based on the calculations for bearing capacity and the information obtained during the slope stability evaluation, we believe that the natural soil will support the proposed construction and will result in suitable factors of safety against bearing capacity type failures.

7. Stability Calculations

The stability of the proposed embankment and landfill was analyzed under several loading conditions. Factors of safety for the embankment and the completed landfill were determined against mass rotational and sliding wedge failures. Static and dynamic (pseudo static) analyses of the embankment and disposal cell were conducted using the configuration as described. Strength parameters used in the stability analysis are listed on Figure 19.

Rotation failure analysis were conducted on the proposed embankment and on the filled landfill cell aided by a computer. The stability program which models this method was developed by Ronald A. Seagull, graduate instructor in research, Purdue University as a joint highway research project in cooperation with the Indiana State Highway Commission.

Stability calculations indicate that the defined embankment and cut/fill section has a static safety factor under long term conditions of approximately 1.5. For the seismic long term conditions, the stability for the embankment alone is calculated to be 1.3.

Calculations indicate that if pore pressures within the foundations soils were increase to a level equivalent to the amount of fill placed for the embankment (end of construction) a static safety factor would be 2.1.

Stability calculations for the final configuration of the landfill indicate a static safety factor of 2.3 with a minimum calculated seismic safety factor of 1.6.

A summary of the safety factors obtained are included on Figure 19 with the critical failure planes indicated.

Recommended minimum factors of safety are dependent on the uncertainty of soils strength parameters and the cost of consequences of slope failure. The Environmental Protection Agency recommends use of minimum static factor of 1.5 for a slope where the cost of repair is comparable to the cost of construction and if there is no danger to human life or other valuable property if the slope fails with large uncertainty in soil strength parameters. The corresponding minimum factor of safety under seismic conditions is 1.3. (Guide to Technical Resources for the Design of Land Disposal Facilities, EPA/625-6-88/018, December 1988, Risk Reduction Engineering Laboratory and Center for Environmental Research Information, Office of Research and Development, USCPA, Cincinnati, Ohio 45628.)

Based on the subsoils encountered, laboratory test results, stability analysis and given loading conditions, the embankment and proposed landfill cell meet the minimum safety factors.

8. Synthetic Slope Stability

Each of the synthetic liner areas contains dissimilar materials or is constructed of dissimilar materials which have significantly different friction factors or resistance to sliding. The weakest interface was evaluated on an infinite slope type of evaluation under both static and pseudo static conditions. Listed below is a table summarizing the location of the synthetic liner system, the weakest friction value, the slope upon which the material is placed and the static and pseudo static factors of safety.

Location	Weakest Interface	Friction (degrees)	Cohesion (psf)	Slope (H:V)	Safety Factor	
					Static	Seismic
Interior Slope	GCL/Soil	26	30	2:1	1.2	1.0
Floor	HDPE/GCL	8	0	1.7%	8	1.3
Cap (Slope)	GCL	18	50	4:1	11	4
Cap (Top)	GCL	18	50	5%	2.2	1.6

Note: The interior slope was evaluated with 20 feet of protective soil cover sloped at 2.5:1.

These results indicate that the synthetic materials, as currently designed, meet the minimum criteria for factors of safety except for the interior 2:1 slopes. The integrity and desired factor of safety may be achieved on the 2:1 slopes by placing the soil protective cover in 10-foot vertical stages or by verifying that the interface strength between the GCL and underlying soil on the slope is greater than we have assumed. The literature indicates that a higher strength will most likely apply. We recommend that the strength of the proposed synthetic materials and the underlying soils be verified prior to construction.

C. Settlement

Based on the subsurface information, along with the anticipated weights of the waste material and configuration of the landfill, the amount of settlement that will likely be experienced by the facility was estimated. Due to the variation in the waste height, along with the anticipated variation and, therefore, compressibility of the foundation soils, we estimate that the total settlement on the upper toe (west end) of the floor of the landfill to be approximately 5 inches with the settlement at the toe at the east end of the facility will be approximately 1 to 2 feet. The variation in settlement will depend on the load and also the subsurface soil conditions. We estimate, however, that this will happen fairly gradually and will not be detrimental to the performance of the liner system.

D. Liquefaction

The density and type of soil encountered during this and Kleinfelder's study indicate that there may be thin, dis-continuous layers of soil that may be subject to liquefaction during a major seismic event.

The locations where the soil is potentially liquefiable, as delineated by Kleinfelder are in the borrow area, and not under the landfill. The subsurface soil investigated during this study was found to not be susceptible to liquefaction at an acceleration with a 5% probability of exceedance within 50 years.

Based on the proposed construction, the existing soil conditions, the depth of ground water, and the increased stress on the underlying soil due to the placement of the waste, it is our professional opinion that the likelihood of liquefaction is very low and would require an acceleration higher than predicted to have a 2 percent probability of exceedance in 50 years.

GCL COMPATIBILITY

Due to the salty environment of the site, tests were conducted in order to verify that the GCL will perform as intended even under adverse conditions of the site.

A sample of bentonite from two different suppliers were obtained and tested for their Atterberg Limits using distilled water, water obtained from a piezometer at the site, along with a water leached from soil obtained from four different locations at the site.

The testing indicates the greatest impact on plasticity of the bentonite to be with water leached through Sample A. Using the Sample A leached water, a permeability test was conducted on the "GSE" bentonite with a permeability of 1.5×10^{-9} cm/sec.

CONSTRUCTION CONSIDERATIONS

Based on the subsurface investigation, the proposed materials and our experience with this type of construction, the following precautions should be observed during design and construction of the proposed landfill.

A. Foundation Preparation

Foundation preparation consists of removing any disturbed soils in the area of proposed construction. Any vegetation or debris that is within the areas to receive fill should be removed. Positive measures should be taken to remove any material in any compactive areas that do not meet the compaction criteria.

B. Embankment Construction

1. Materials

The embankment may be constructed with a mixture of clay, silt, sand or gravel soils. This indicates that any of the soil encountered at the site would be potentially suitable.

Materials for construction of the embankment are available from the surrounding area.

2. Compaction

All fill within the embankment should be placed and compacted to at least 95 percent of the maximum dry density as determined by ASTM D-698. Moisture content of the fill would be at or above optimum moisture content to facilitate the compaction process.

Fill should be placed in uniform lifts not more than 8 inches thick prior to compaction. Compaction should be accomplished with heavy compaction equipment.

Lifts compacted by hand operated equipment should be no more than 4 inches in loose thickness.

3. Benching

Fill placed on slopes steeper than 5:1 (horizontal to vertical) should be benched into the slope with benches no greater than 2 feet. In areas where the slope is irregular and in rock, the need for benching may be eliminated.

4. <u>Erosion Protection</u>

Exterior portions of the embankment may be protected to reduce erosion or repaired when needed.

5. <u>Construction Quality Control</u>

The materials are to be observed and tested by a representative of the soils engineer to verify that the densities and moisture contents meet the project specifications.

LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical engineering practices in the area for the use of the client for design purposes. The conclusions and recommendations included within the report are based on the information obtained from the borings drilled at the approximate locations indicated on the site plan and the data obtained from laboratory testing. Variations in the subsurface conditions may not become evident until additional exploration or excavation is conducted. If the subsurface conditions or groundwater level are found to be significantly different from those described above, we should be notified to reevaluate our recommendations.

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, P.C.

REGISTERED PROFESSIONAL ENGINEER
No. 143916
JAMES E.
NORDQUIST
6/15/05
STATE OF UTAH

James E. Nordquist, P.E.

JEN/sc

REFERENCES

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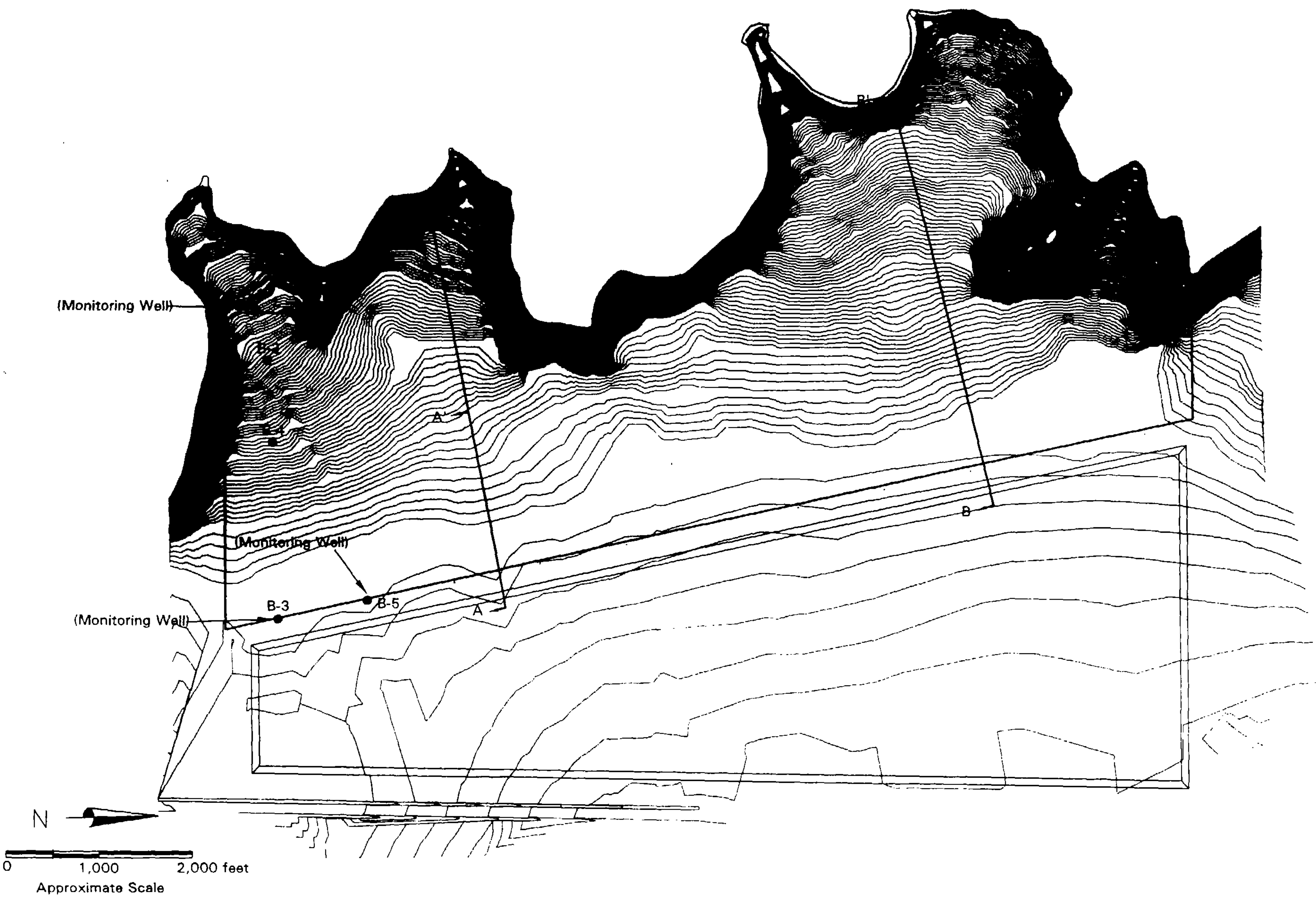
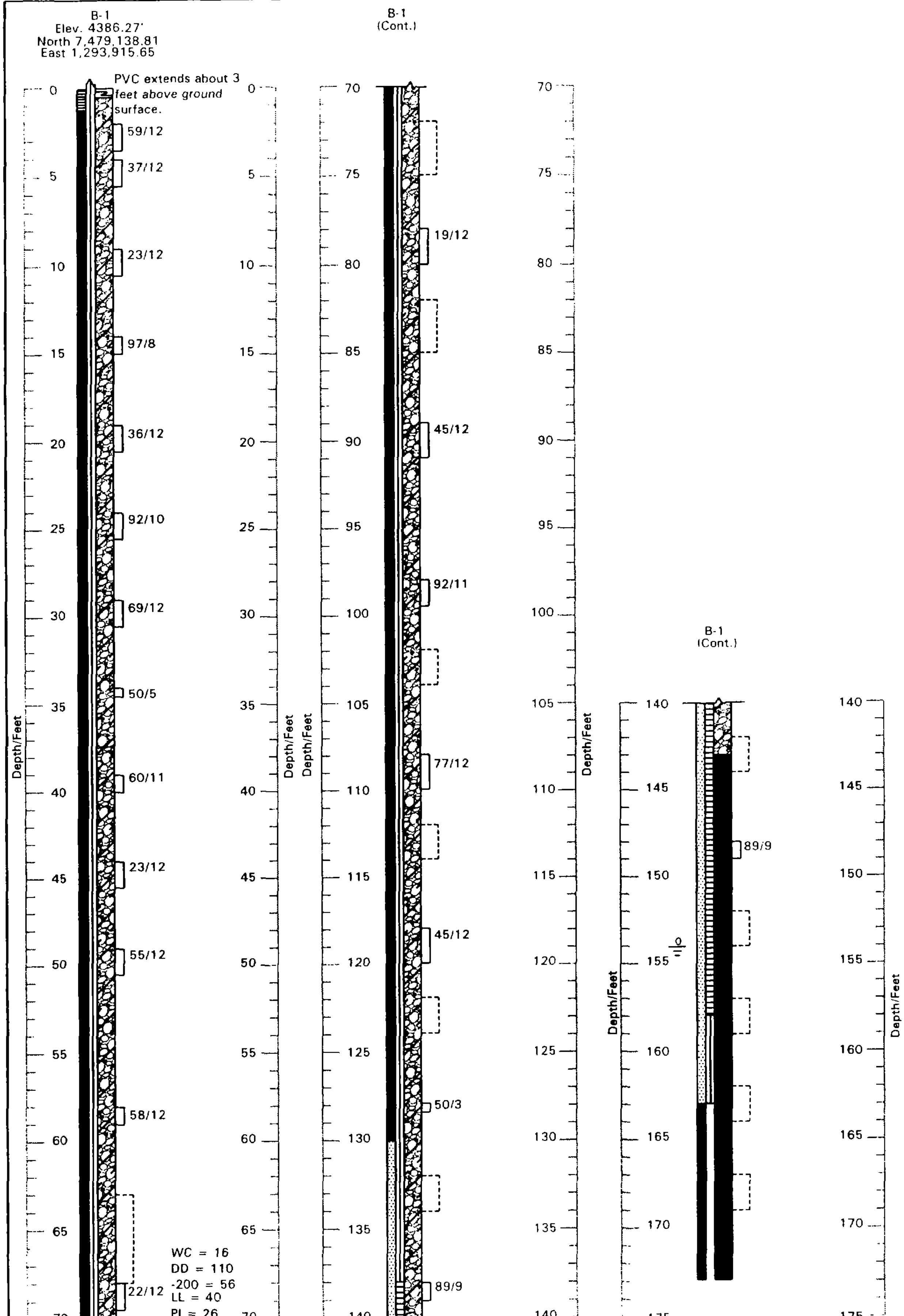


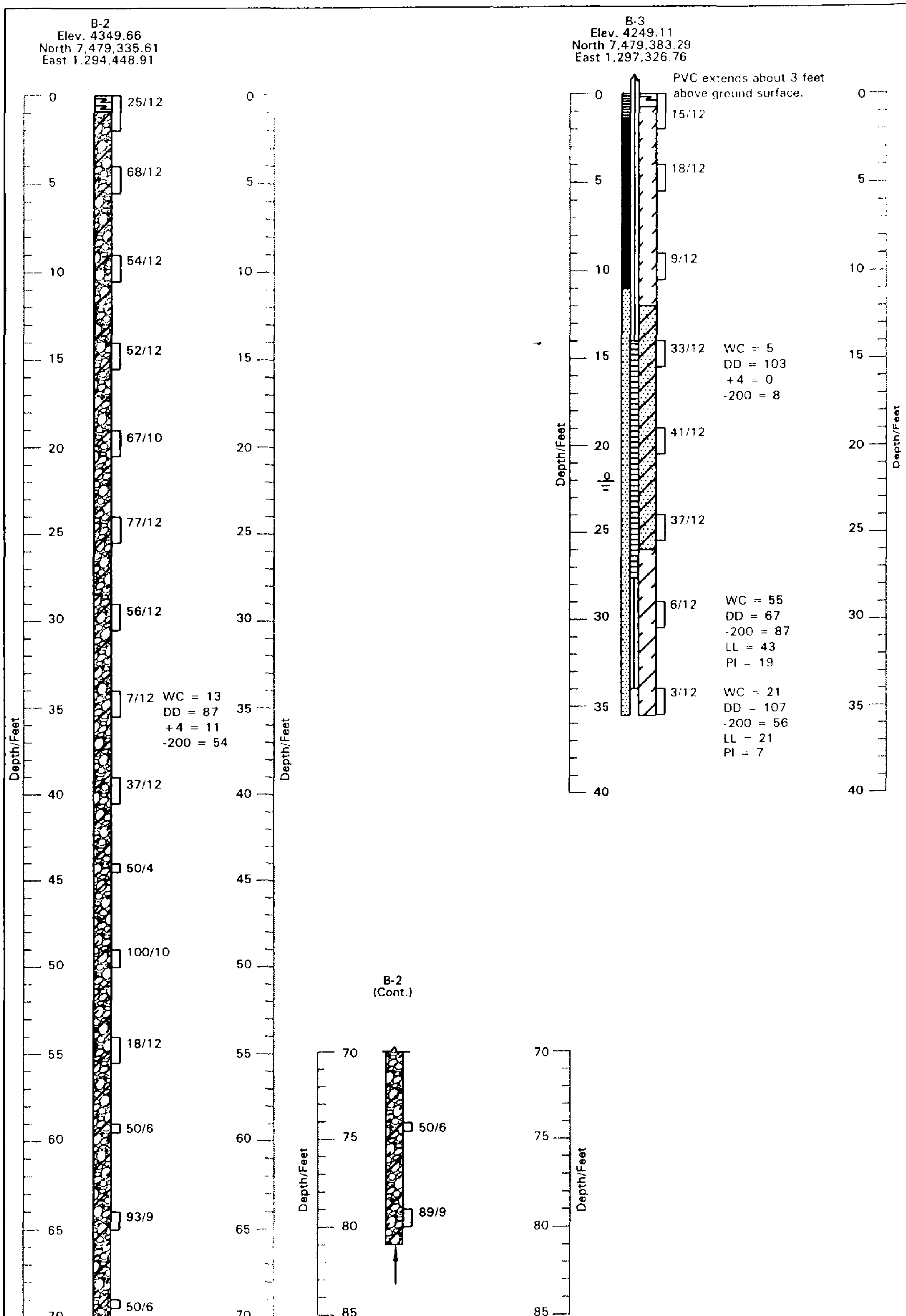
1040644	AGEC	*Locations of Exploratory Borings*	Figure 1



Approximate Vertical Scale 1" = 8'

See Figure 5 for Legend and Notes

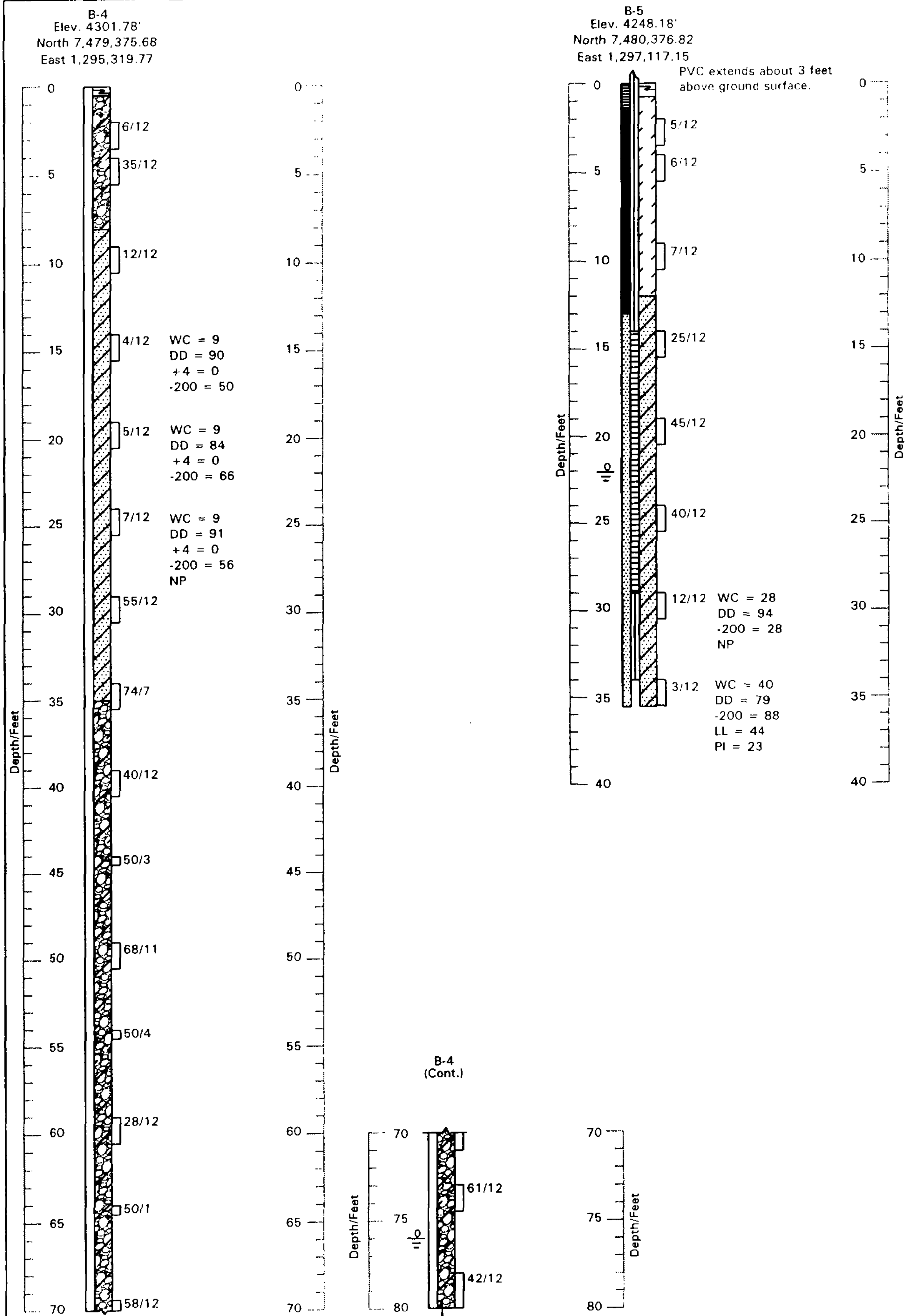
1040644	AGEC	Logs of Exploratory Borings	Figure 2



Approximate Vertical Scale 1" = 8'

See Figure 5 for Legend and Notes

1040644	AGRA	Logs of Exploratory Borings	Figure 3



Approximate Vertical Scale 1" = 8'

See Figure 5 for Legend and Notes

1040644	AGEC	Logs of Exploratory Borings	Figure 4

LEGEND:

Topsoil;

Lean Clay (CL); interlayered with sandy silt, stiff to very stiff, slightly moist to moist, brownish gray.

Silty Clay (CL-ML); sandy, medium to soft, wet, gray.

Sand (SM); silty, occassional lean clay layers, loose to dense, moist to wet, gray to grayish brown.

Gravel (GM/GC); sandy, silty and clayey, occassional cobble and boulders, medium to very dense, moist, brownish gray.

Gray Limestone

10/12 California Drive sample taken. The symbol 10/12 indicates that 10 blows from a 140 pound automatic hammer falling 30 inches were required to drive the sampler 12 inches.

Indicates disturbed sample taken.

Indicates slotted 1 ½ inch PVC pipe installed in the boring to the depth shown.

0 Indicates the depth to free water and the number of days after drilling the measurement was taken.

Indicates screened portion of monitoring well. Screen slots 0.010 inches.

Indicates solid 2" diameter PVC pipe.

Indicates annular space backfilled with Portland Cement Concrete.

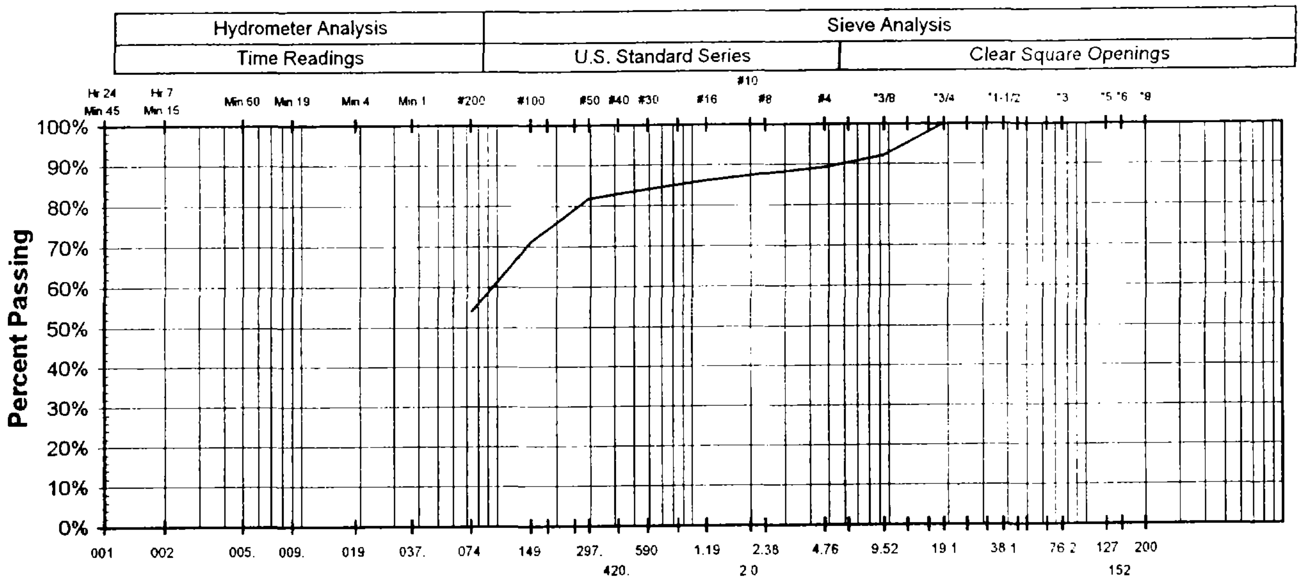
Indicates annular space backfilled with bentonite.

Indicates annular space backfilled with sand.

NOTES:

1. Borings were drilled on October 13, 14, 15, 18, 20, 21, 22, 25, 26, 27, 28 and 29, 2004 with 8-inch diameter hollow-stem auger and 3.5 inch tri-cone bit with air circulation.
2. Locations of borings were provided by civil engineer.
3. Elevations of borings were measured by civil engineer.
4. The boring locations and elevations should be considered accurate only to the degree implied by the method used.
5. The lines between the materials shown on the boring logs represent the approximate boundaries between material types and the transitions may be gradual.
6. Water level readings shown on the logs were made at the time and under the conditions indicated. Fluctuations in the water level may occur with time.
7. Monitor wells were completed with a 4 inch square steel locking cover set in a 2 foot square concrete slab. The 2-inch diameter PVC pipe protected by the well cover extends to approximately 3 feet above the ground surface.
8. WC = Water Content (%);
   DD = Dry Density (pcf);
   +4 = Percent Retained on No. 4 Sieve;
   -200 = Percent Passing No. 200 Sieve;
   LL = Liquid Limit (%);
   PI = Plasticity Index (%);
   NP = Non Plastic

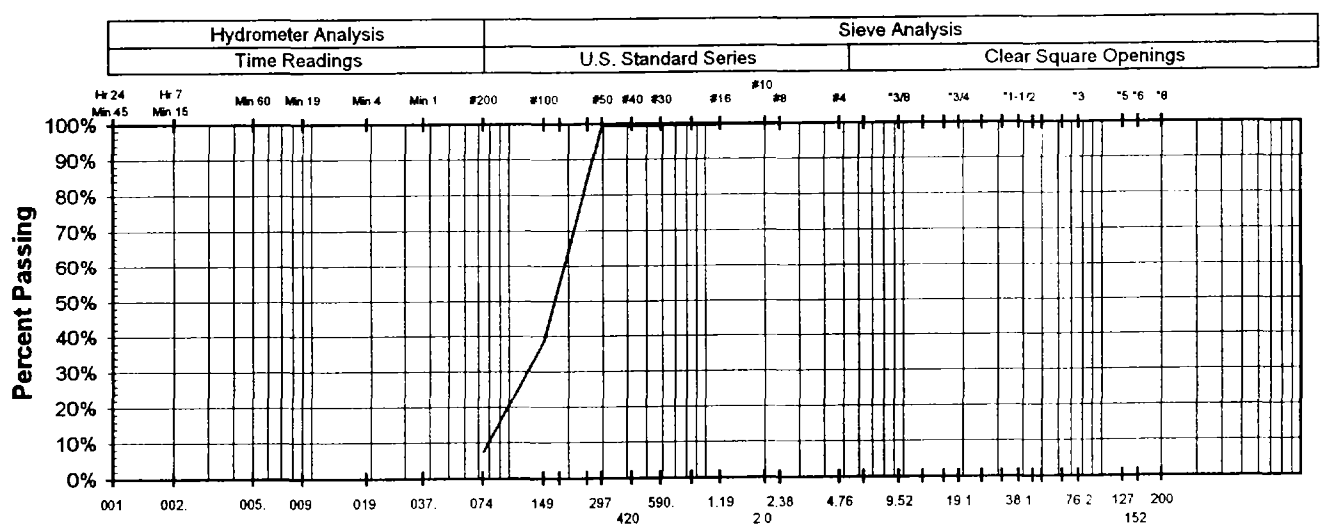
APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.



Diameter of Particle in Millimeters

Clay to Silt	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

Gravel	11%	Liquid Limit	-
Sand	35%	Plasticity Index	-
Silt and Clay	54%	Sample Location	B-2 @ 34'
Sample Description	Sandy Silt		

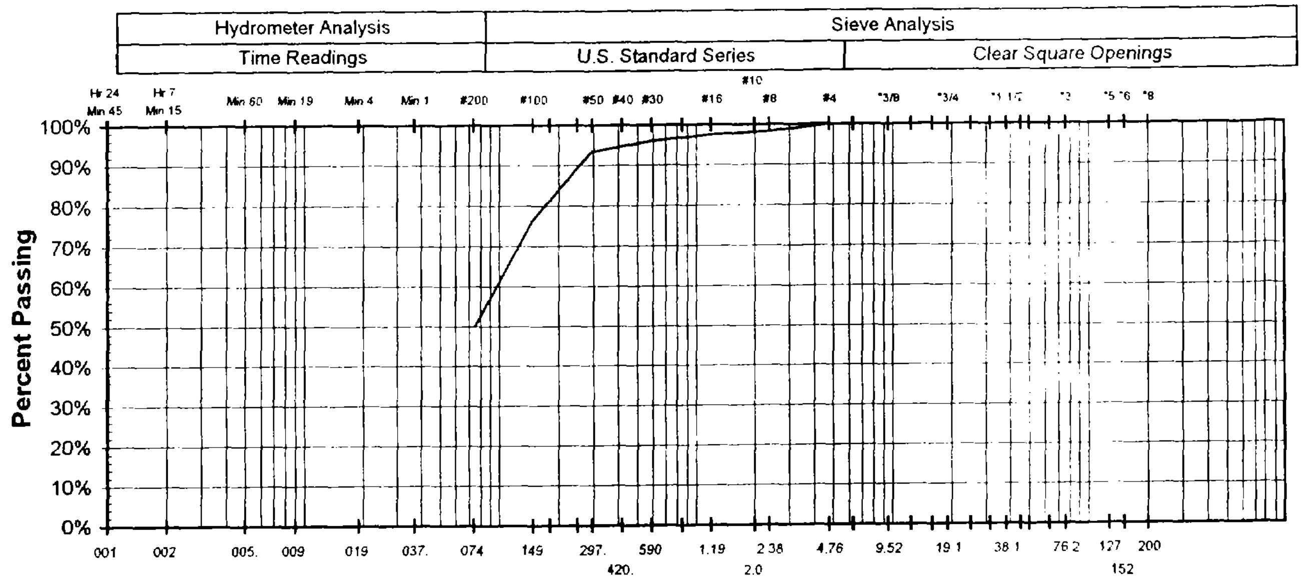


Diameter of Particle in Millimeters

Clay to Silt	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

Gravel	0%	Liquid Limit	-
Sand	92%	Plasticity Index	-
Silt and Clay	8%	Sample Location	B-3 @ 14'
Sample Description	Poorly Graded Sand with Silt		

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, INC.

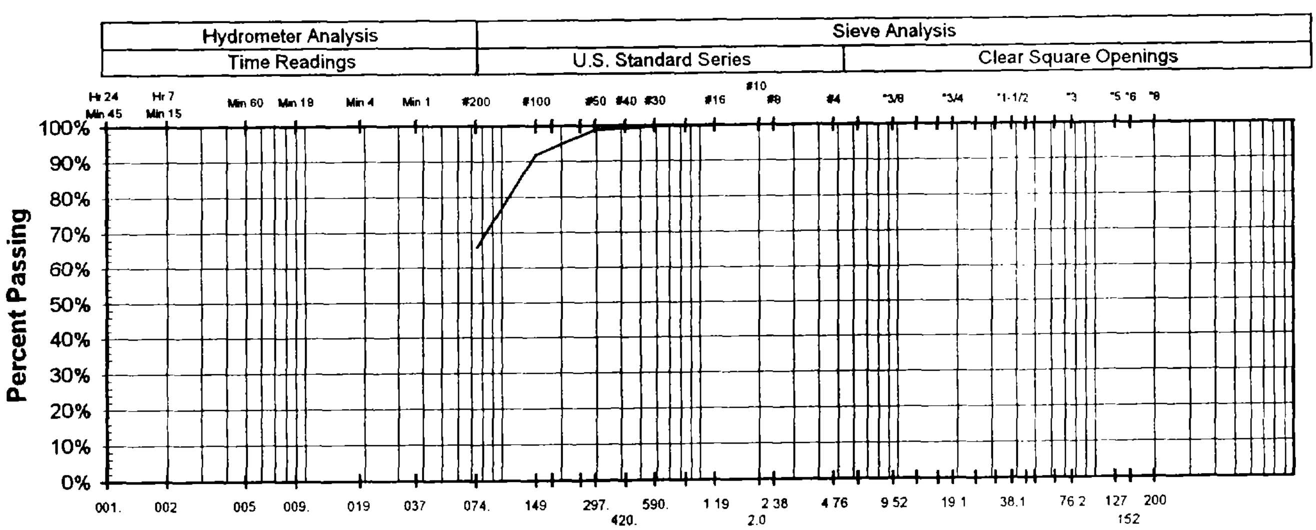


Diameter of Particle in Millimeters

Clay to Silt	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

Gravel 0%
Sand 50%
Silt and Clay 50%
Sample Description Sandy Silt

Liquid Limit -
Plasticity Index -
Sample Location B-4 @ 14'



Diameter of Particle in Millimeters

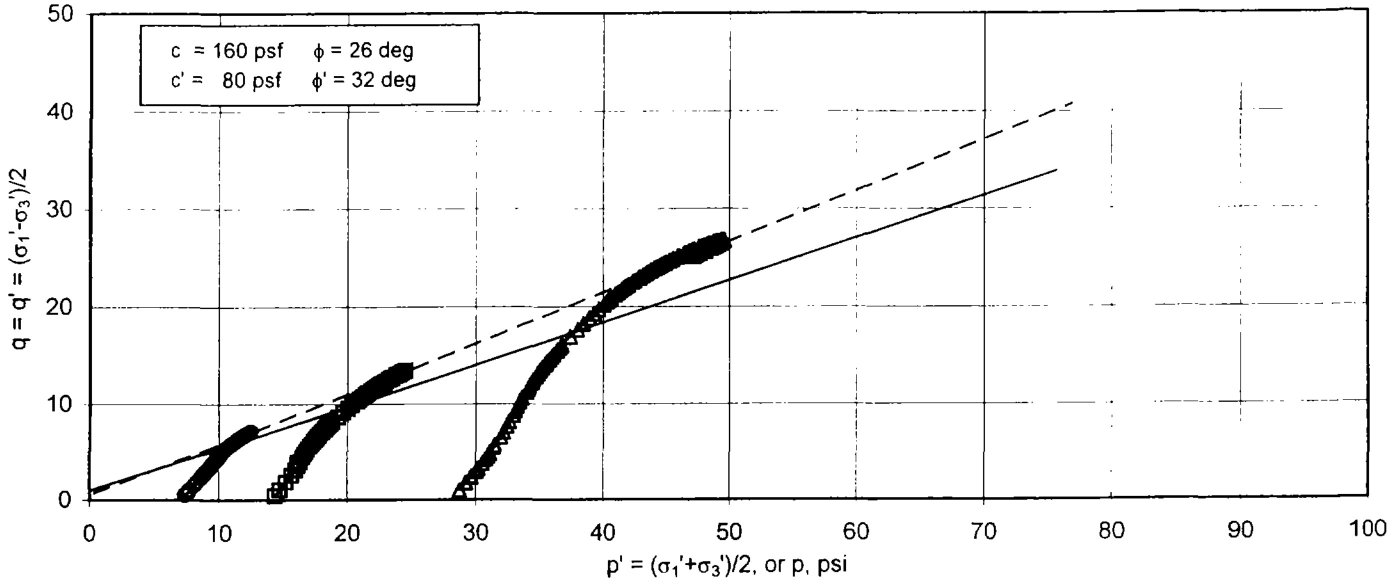
Clay to Silt	Sand			Gravel		Cobbles	Boulders
	Fine	Medium	Coarse	Fine	Coarse		

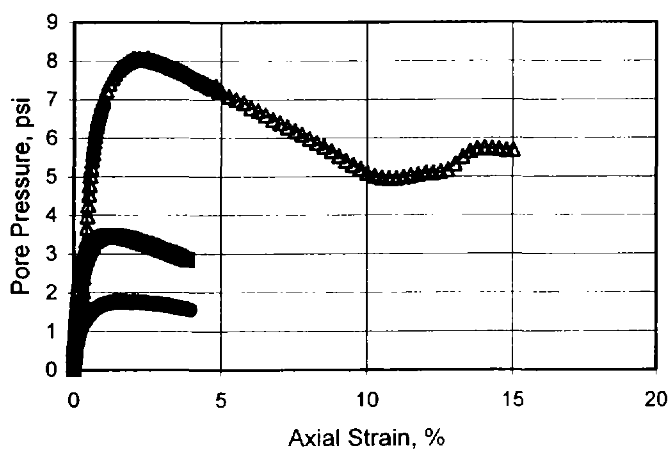
Gravel 0%
Sand 34%
Silt and Clay 66%
Sample Description Sandy Silt

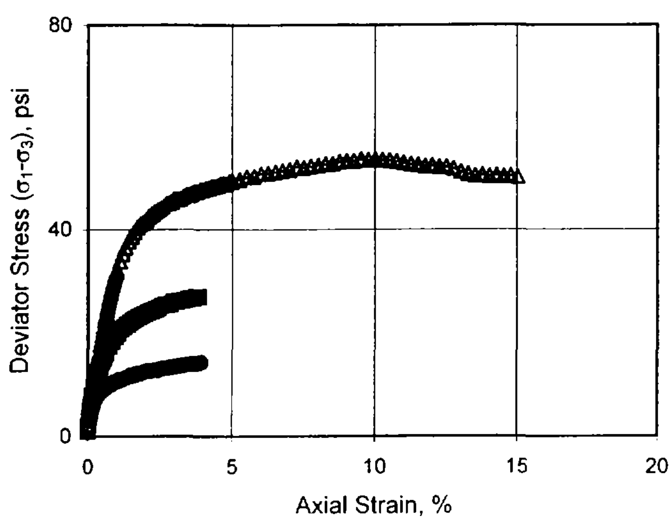
Liquid Limit -
Plasticity Index -
Sample Location B-4 @ 19'

Project No. 1040644 **GRADATION TEST RESULTS** Figure 7

Applied Geotechnical Engineering Consultants, P.C.







Test No. (Symbol)	O	□	Δ
Sample Type	undisturbed		
Length, in.	4.00	3.83	3.72
Diameter, in.	1.93	1.76	1.65
Dry Density, pcf	91	N/A	N/A
Moisture Content, %	9	N/A	N/A
Consolidation Pressure, psi	6.9	13.9	27.8
"B" Parameter	0.96	0.96	0.96
Total Confining Stress (σ_3), psi	6.9	13.9	27.8
Total Axial Stress (σ_1), psi	20.3	39.9	73.7
Deviator Stress (σ_1-σ_3), psi	13.4	26.0	45.9
Effective Lateral Stress (σ_3'), psi	5.2	10.8	19.9
Effective Axial Stress (σ_1'), psi	18.6	36.8	65.8
Pore Pressure (μ), psi	1.7	3.1	7.9
Strain, %	3.0	3.0	3.0
Remarks	Multistage Test (CU) Consolidated		
Undrained with pore pressure measurements.			
Sample saturated with back pressure saturation.			

Sample Index Properties	
Natural Dry Density, pcf	91
Natural Moisture Content, %	9
Liquid Limit, %	
Plasticity Index, %	non-plastic
Percent Gravel	0
Percent Sand	44
Percent Passing No. 200 Sieve	56

Sample Description: Sandy Silt

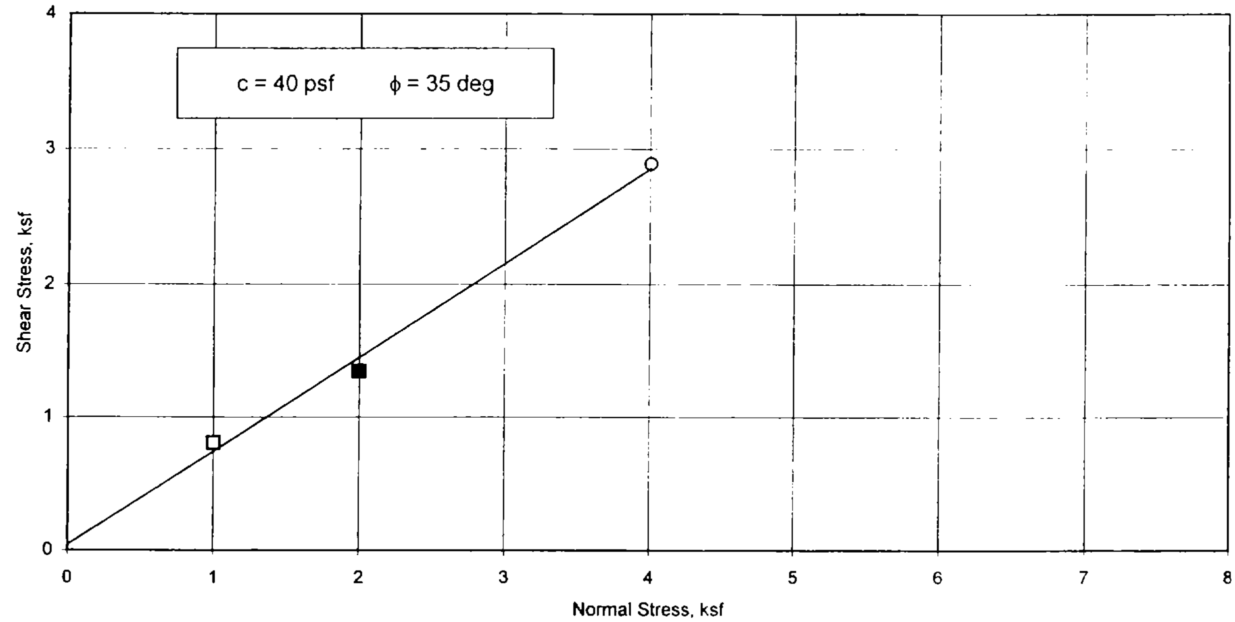
Sample Location: B-4 @ 24'

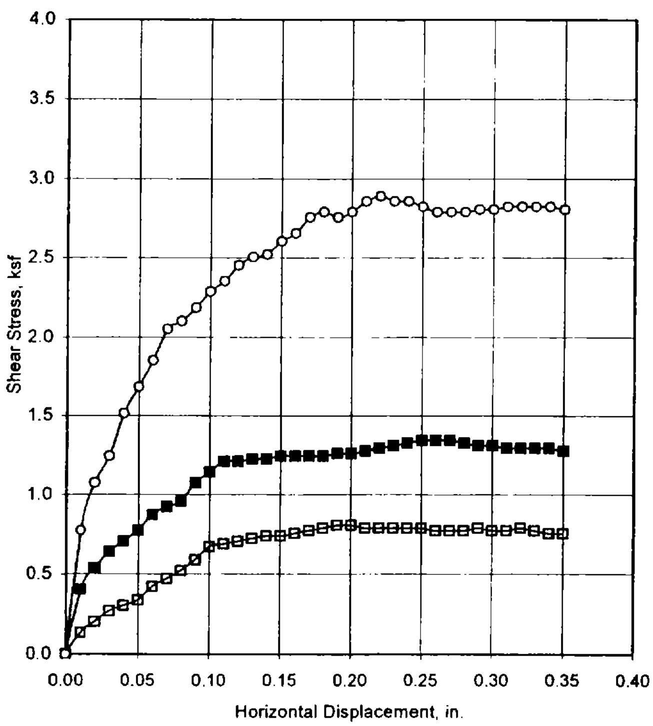
Project No. 1040644

Triaxial Compression Test Results

Figure 8

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(○)
Sample Type	Undisturbed		
Length, in.	1.00	1.00	1.00
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0.81	1.35	2.89
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	87
Moisture Content, %	13
Liquid Limit, %	
Plasticity Index, %	
Percent Gravel	11
Percent Sand	35
Percent Passing No. 200 Sieve	54

Type of Test: Consolidated Wetted

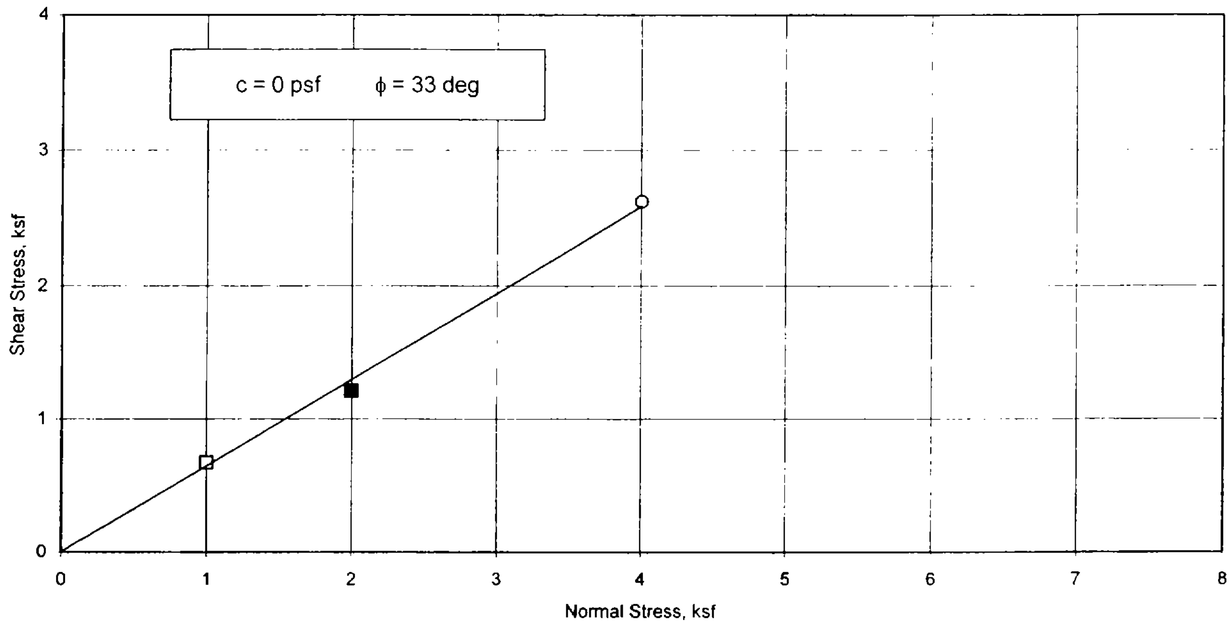
Sample Description: Sandy Silt   From: B-2 @ 34'

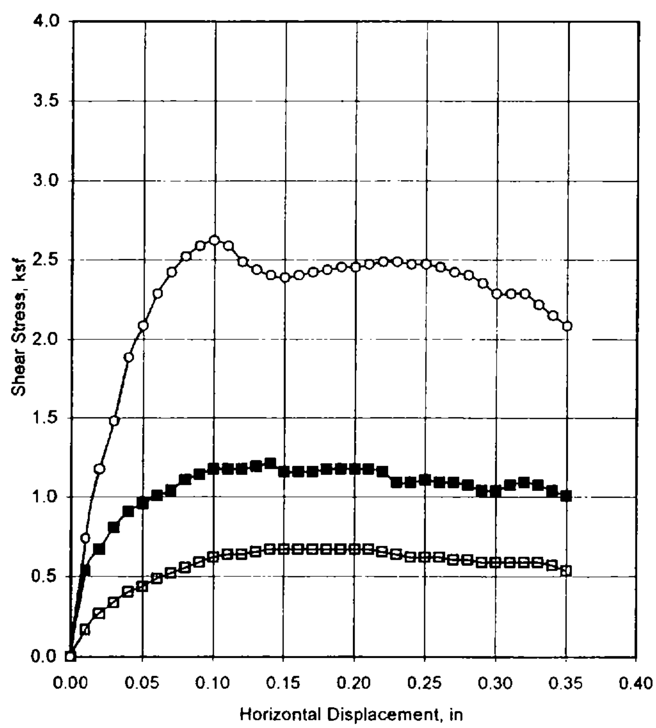
Project No. 1040644

Direct Shear Test Results

Figure 9

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(○)
Sample Type	Undisturbed		
Length, in.	1.00	1.00	1.00
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1.0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0.67	1.21	2.62
Remarks	Strain Rate 0.05 in/min		

Sample Index Properties	
Dry Density, pcf	103
Moisture Content, %	5
Liquid Limit, %	
Plasticity Index, %	
Percent Gravel	0
Percent Sand	92
Percent Passing No. 200 Sieve	8

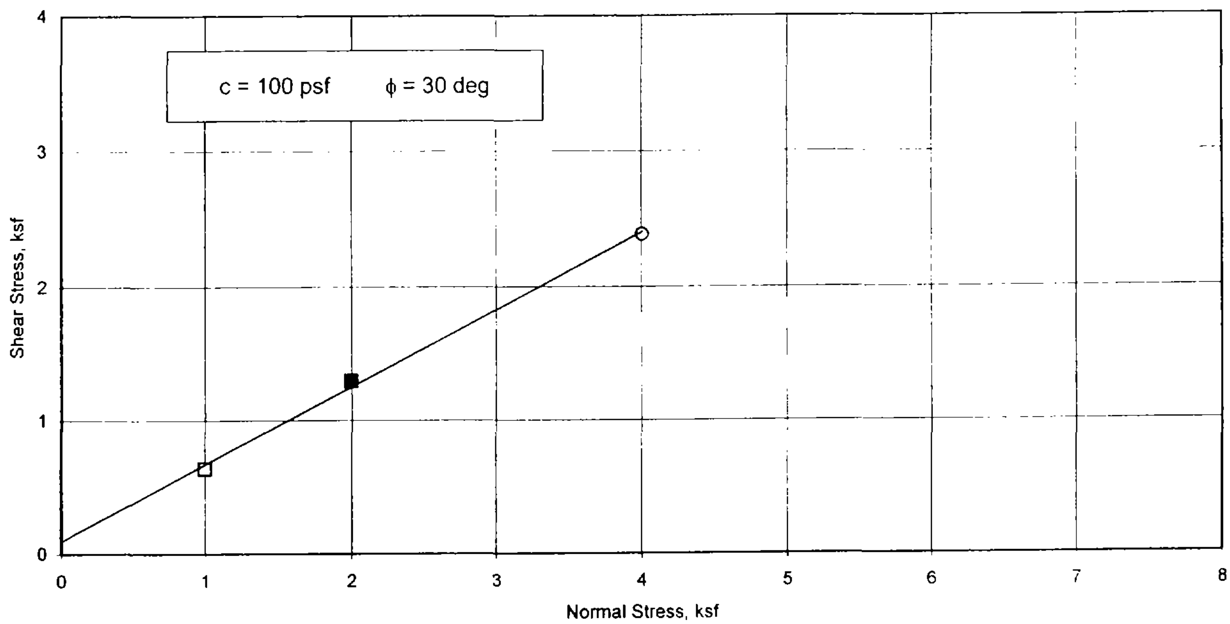
Type of Test: Consolidated Wetted
Sample Description: Poorly Graded Sand with Silt   From: B-3 @ 14'

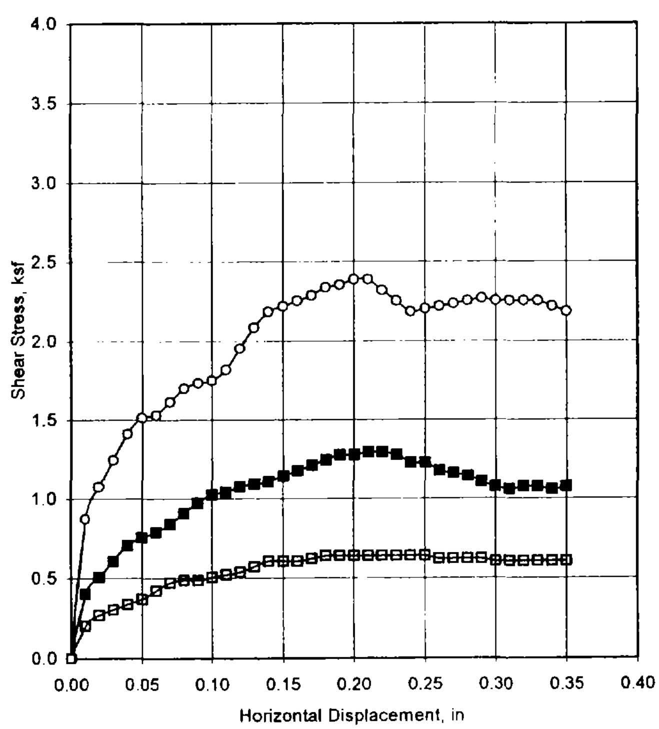
Project No. 1040644

Direct Shear Test Results

Figure 10

Applied Geotechnical Engineering Consultants, Inc.





Test No. (Symbol)	1(□)	2(■)	3(○)
Sample Type	Undisturbed		
Length, in.	1.00	1 00	1.00
Diameter, in.	1.93	1.93	1.93
Dry Density, pcf	N/A	N/A	N/A
Moisture Content, %	N/A	N/A	N/A
Consolidation Load, ksf	1 0	2.0	4.0
Normal Load, ksf	1.0	2.0	4.0
Shear Stress, ksf	0 64	1.29	2.39
Remarks	Strain Rate 0.05 in/min.		

Sample Index Properties	
Dry Density, pcf	90
Moisture Content, %	9
Liquid Limit, %	
Plasticity Index, %	
Percent Gravel	0
Percent Sand	50
Percent Passing No. 200 Sieve	50

Type of Test: Consolidated Wetted

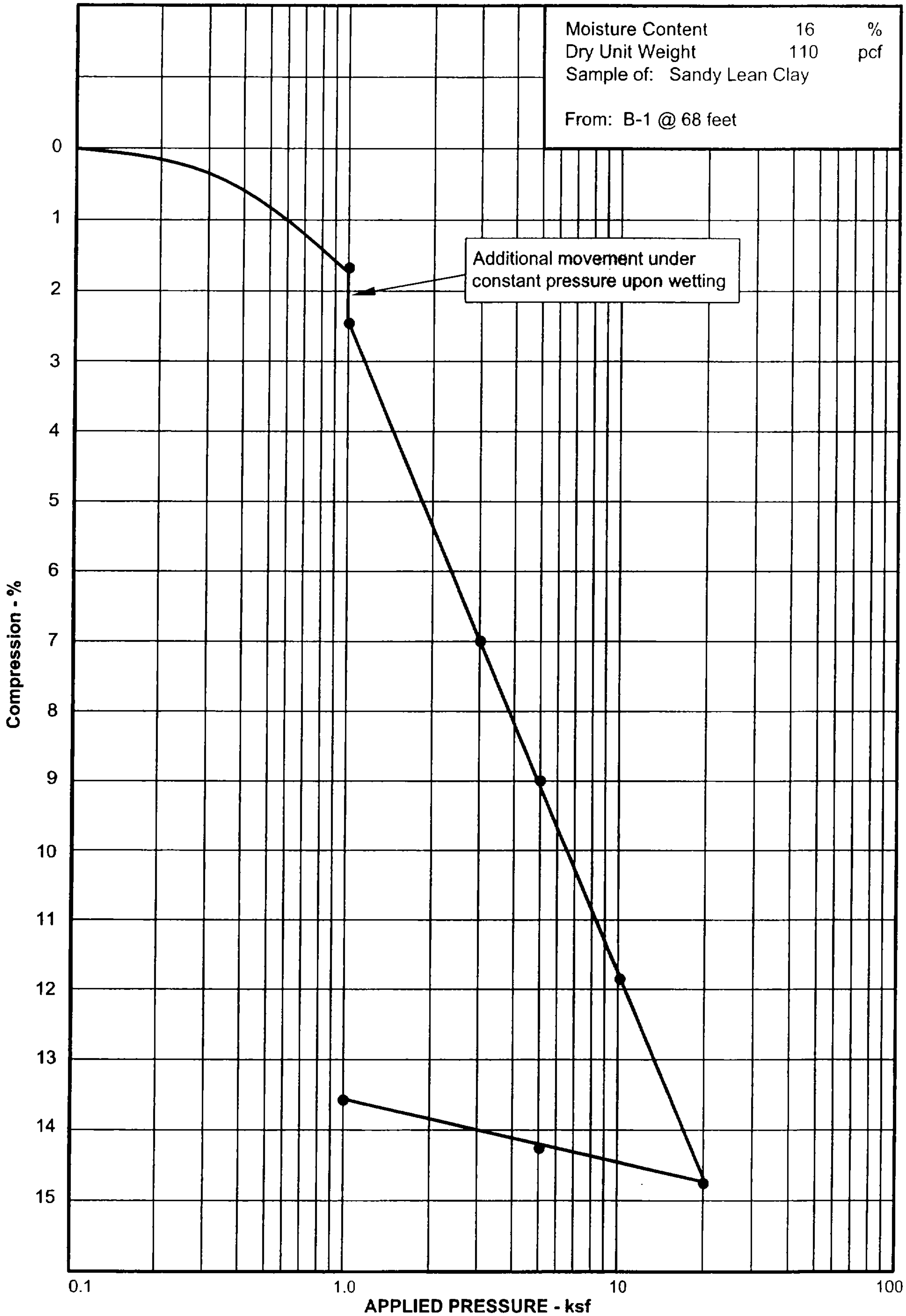
Sample Description: Sandy Silt From: B-4 @ 14'

Project No. 1040644

Direct Shear Test Results

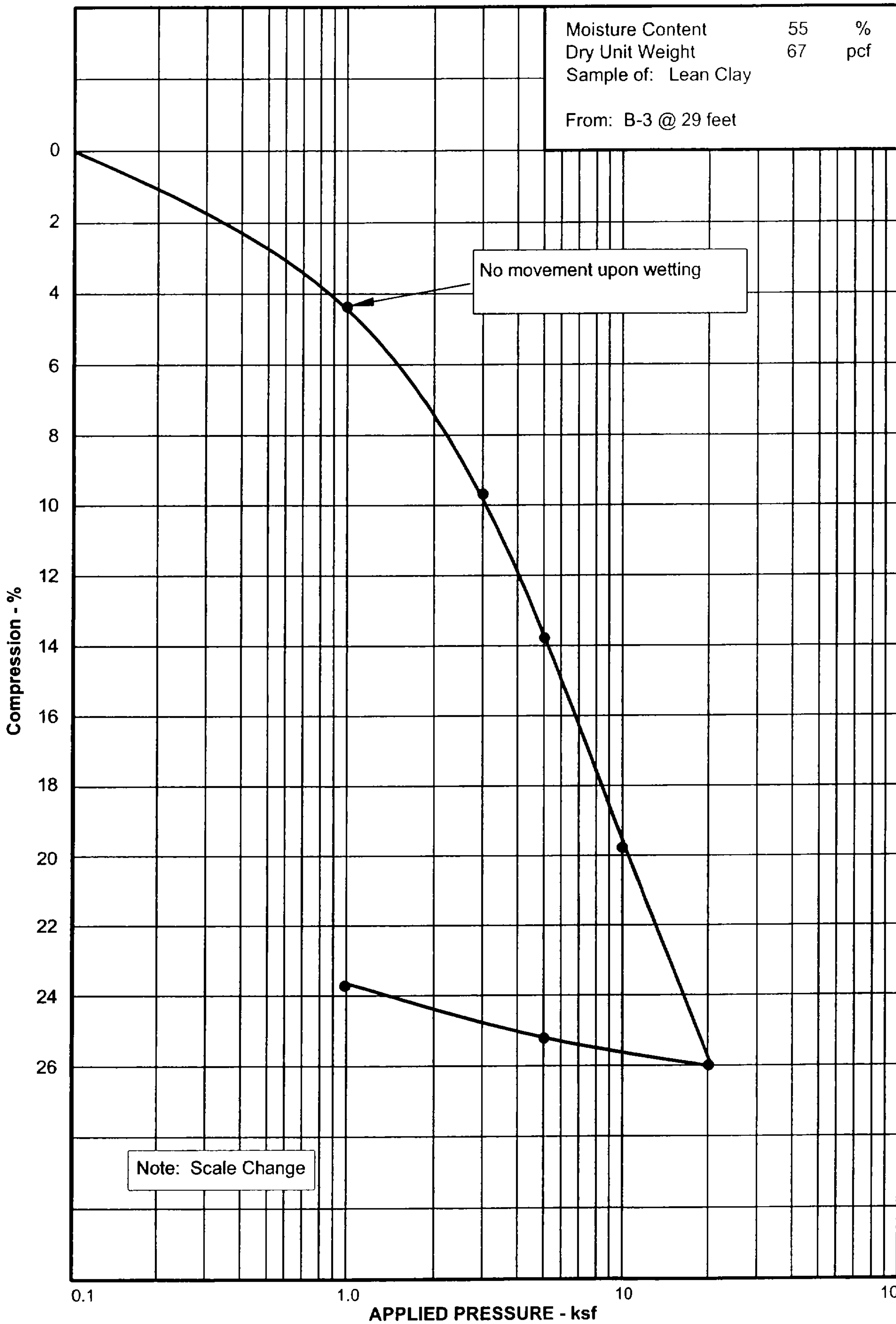
Figure 11

Applied Geotechnical Engineering Consultants, P.C.



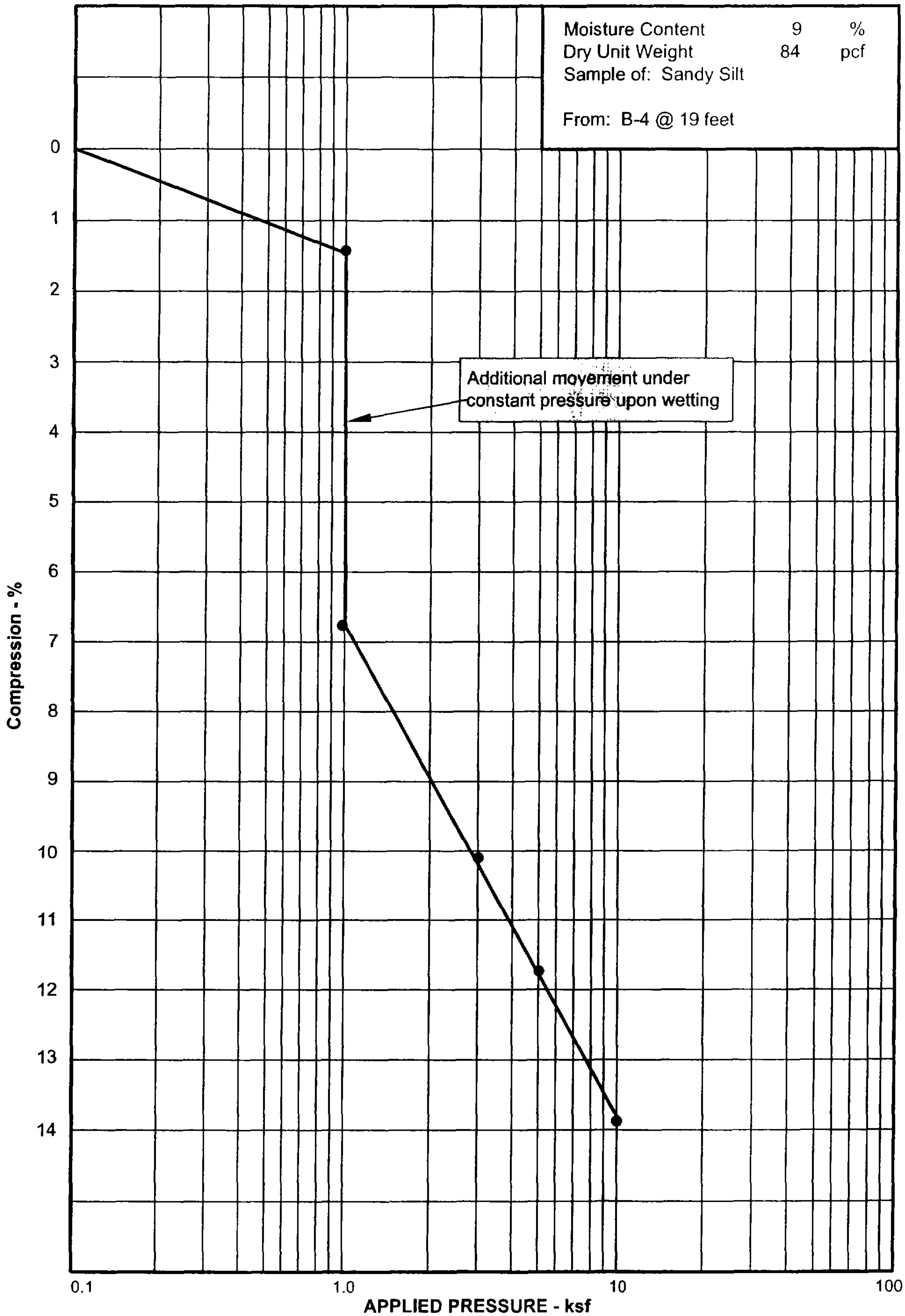
Project No. 1040644 **CONSOLIDATION TEST RESULTS** **Figure 12**

Applied Geotechnical Engineering Consultants, P.C.



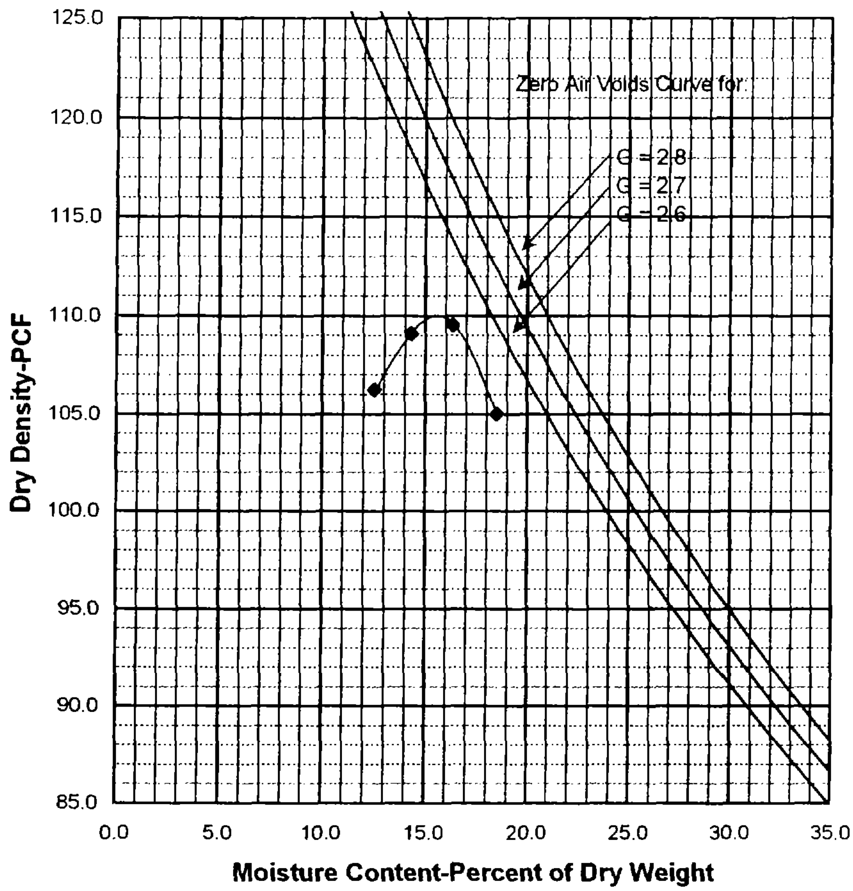
Project No. 1040644 CONSOLIDATION TEST RESULTS Figure 13

Applied Geotechnical Engineering Consultants, P.C.



Project No. 1040644 **CONSOLIDATION TEST RESULTS** **Figure 14**

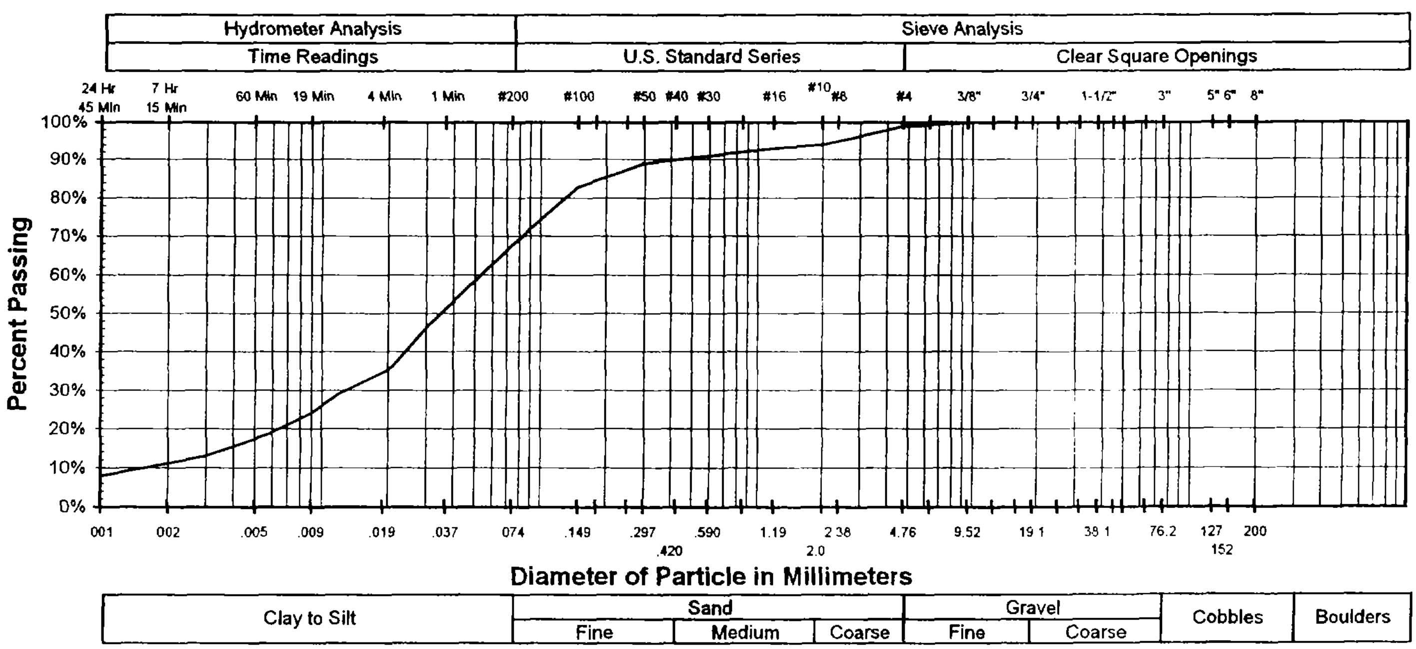
APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, P.C.



Project	Wasatch Regional	
Project No.	1040644	
Sample No.	A	
Maximum Dry Density		110 pcf
Optimum Moisture		15.5%
Atterberg Limits		
Liquid Limit		22%
Plasticity Index		6%
Gradation		
Gravel		1%
Sand		60%
Silt & Clay		39%

Reviewed By:	JS
Test Procedure:	ASTM D698 A
Sample Location:	NW Corner

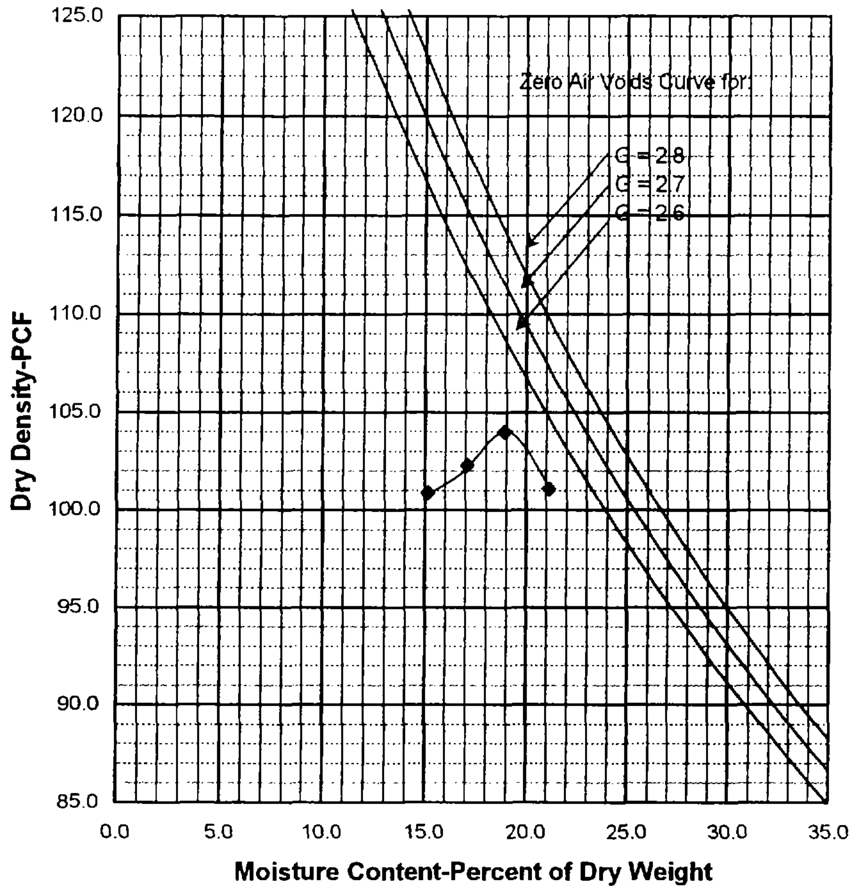
Description: Silty Clayey Sand



GRADATION & MOISTURE-DENSITY RELATIONSHIP

Figure 15

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, P.C.



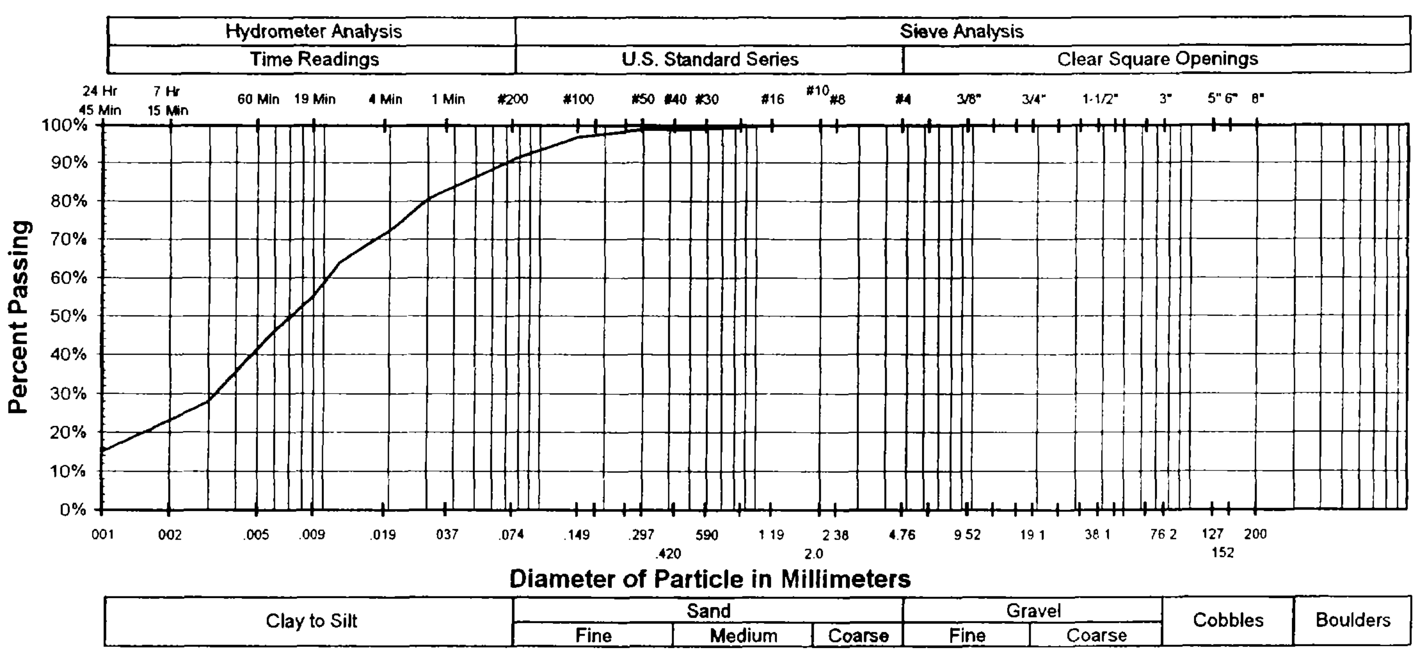
Project	Wasatch Regional	
Project No.	1040644	
Sample No.	B	
Maximum Dry Density		104 pcf
Optimum Moisture		19%
Atterberg Limits		
Liquid Limit		18%
Plasticity Index		1%
Gradation		
Gravel		0%
Sand		9%
Silt & Clay		91%

Reviewed By: JS

Test Procedure: ASTM D698 A

Sample Location: Midpoint South Side

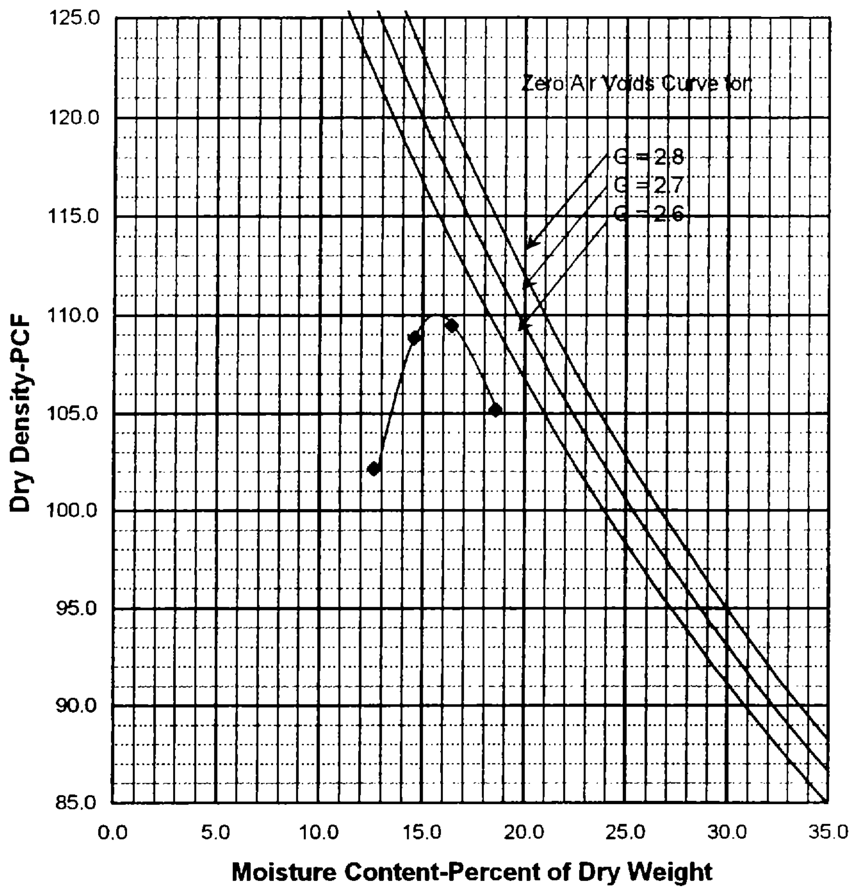
Description: Silt



GRADATION & MOISTURE-DENSITY RELATIONSHIP

Figure 16

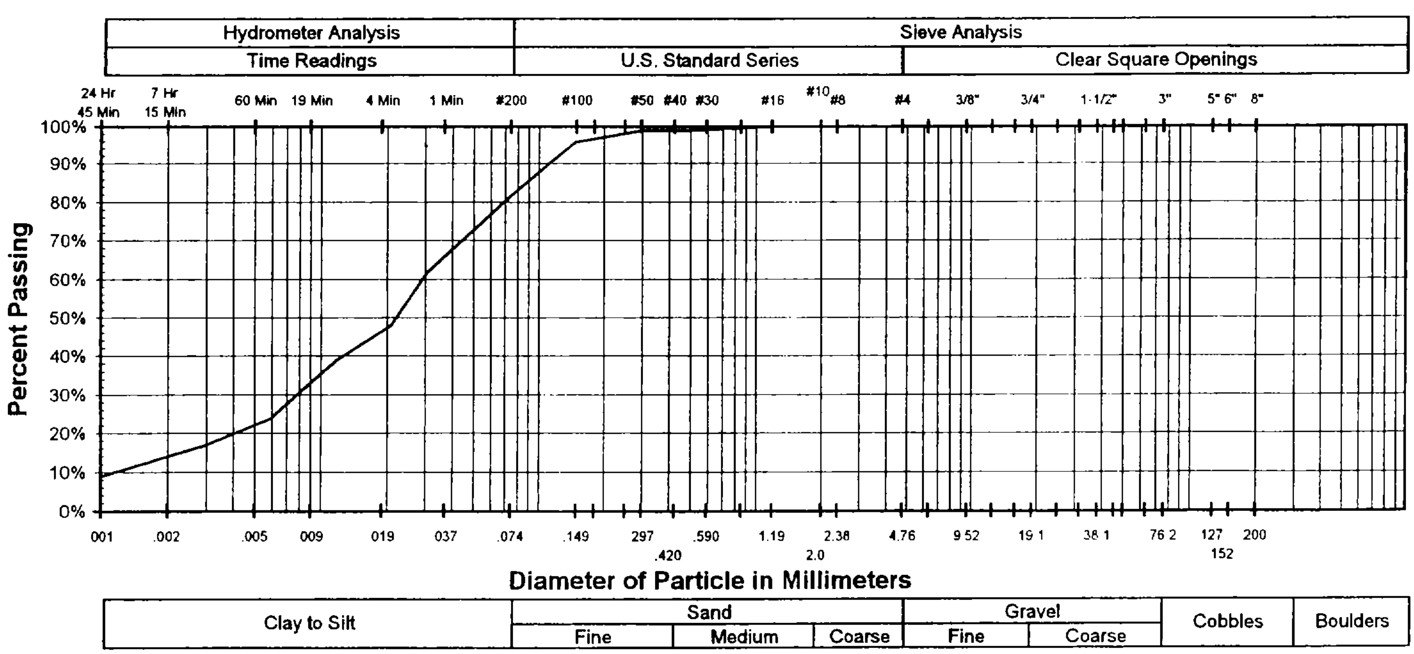
APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, P.C.



Project	Wasatch Regional	
Project No.	1040644	
Sample No.	C	
Maximum Dry Density		110 pcf
Optimum Moisture		15.5%
Atterberg Limits		
Liquid Limit		22%
Plasticity Index		6%
Gradation		
Gravel		0%
Sand		18%
Silt & Clay		82%

Reviewed By: JS
Test Procedure: ASTM D698 A
Sample Location: B-3

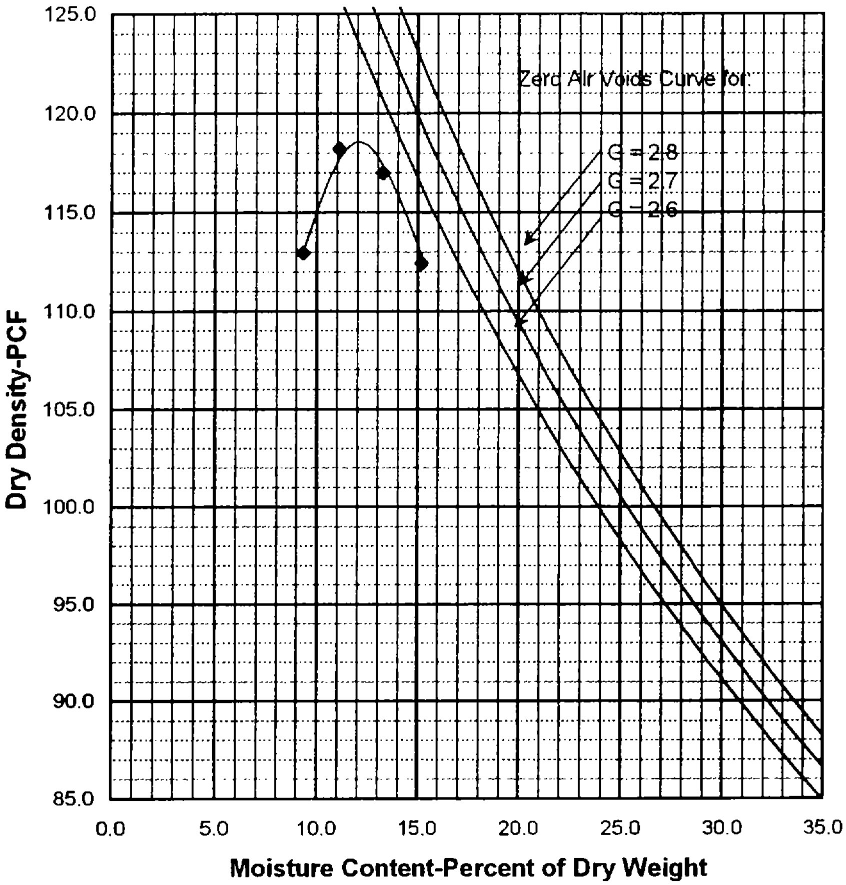
Description: Silty Clay with Sand



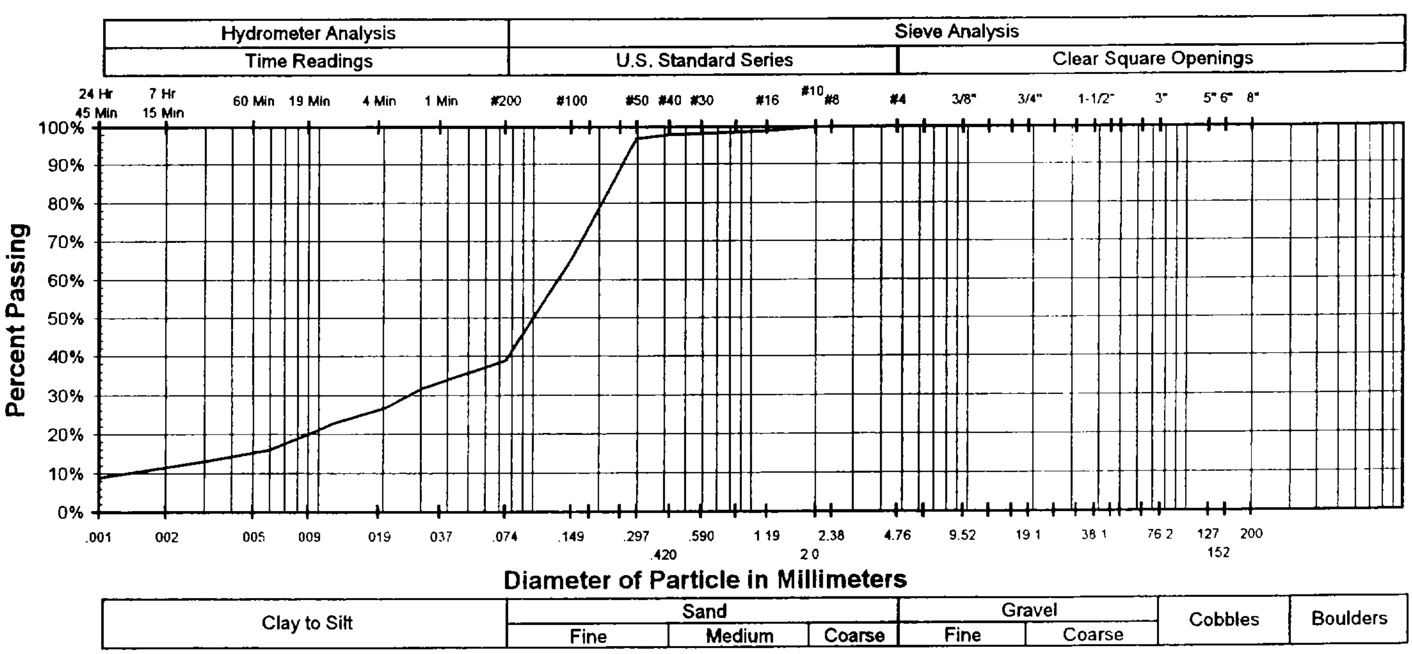
GRADATION & MOISTURE-DENSITY RELATIONSHIP

Figure 17

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, P.C.

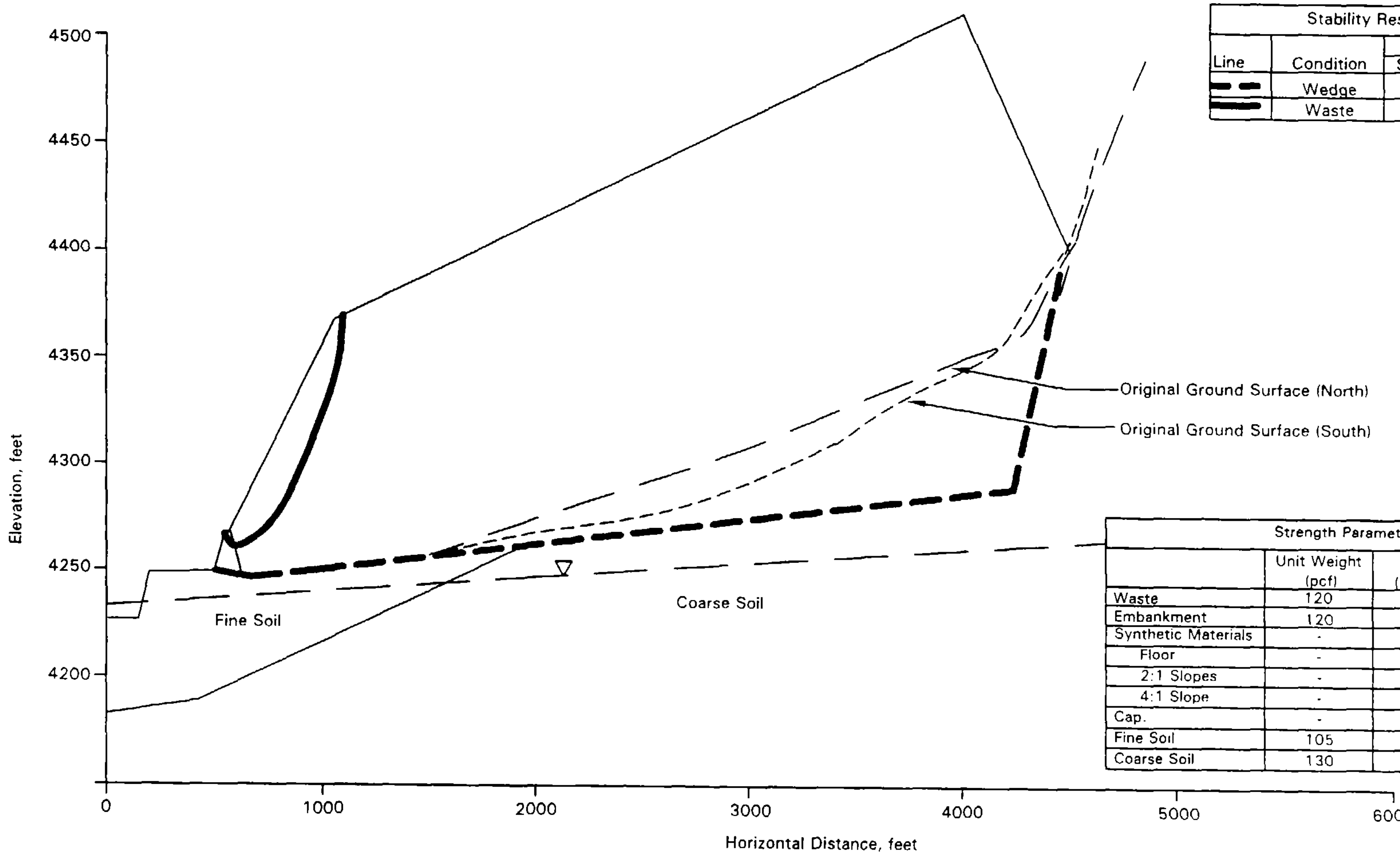


Project	Wasatch Regional
Project No.	1040644
Sample No.	D
Maximum Dry Density	118.5 pcf
Optimum Moisture	12%
Atterberg Limits	
Liquid Limit	17%
Plasticity Index	2%
Gradation	
Gravel	0%
Sand	61%
Silt & Clay	39%
Reviewed By:	JS
Test Procedure:	ASTM D698 A
Sample Location:	B-3
Description:	Silty Sand



GRADATION & MOISTURE-DENSITY RELATIONSHIP

Figure 18



Stability Results			
Line	Condition	Safety Factor	
		Static	Seismic
▬ ▬	Wedge	7.1	1.4
▬▬	Waste	2.3	1.6

Strength Parameters			
	Unit Weight (pcf)	Friction (degrees)	Cohesion (pcf)
Waste	120	25	100
Embankment	120	32	300
Synthetic Materials	-		
Floor	-	8	0
2:1 Slopes	-	23.9	95
4:1 Slope	-	23.9	95
Cap.	-	21.4	84
Fine Soil	105	31	40
Coarse Soil	130	37	0

1040644	ARES	Stability Section A-A" and B-B'	Figure 19

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, P.C.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

PROJECT NUMBER 1040644

SAMPLE LOCATION		NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (PCF)	GRADATION			ATTERBERG LIMITS		UNCONFINED COMPRESSIVE STRENGTH (PSF)	WATER SOLUBLE SULFATE (ppm)	SAMPLE CLASSIFICATION
BORING/ TEST PIT	DEPTH (FEET)			GRAVEL (%)	SAND (%)	SILT/ CLAY (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
B-1	68	16	110			56	40	26			Sandy Lean Clay
B-2	34	13	87	11	35	54					Sandy Silt
B-3	14	5	103	0	92	8					Poorly Graded Sand with Silt
	29	55	67			87	43	19			Lean Clay
	34	21	107			56	21	7			Sandy Silty Clay
B-4	14	9	90	0	50	50					Sandy Silt
	19	9	84	0	34	66					Sandy Silt
	24	9	91	0	44	56		NP			Sandy Silt
B-5	29	28	94			28		NP			Silty Sand
	34	40	79			88	44	23			Lean Clay

APPENDIX 1

Soil Characteristics

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/9/04 BY [illegible]

SUBJECT Soil Characteristics SHEET 1 OF 6

Kleinfelder Study (May 18, 2004 - Proj. #35467.003

Compressibility

Boring	Depth	Cr'	Cc'	Mpp	Description
B-2	5'	0.018	0.177	900	Lean Clay w/Sand
B-3	7½'	0.014	0.065	7000	Sandy Lean Clay
B-4	15'	0.022	0.064	2000	Sandy Lean Clay
B-5	7½'	0.007	0.108	5000	Sandy Silty Clay
B-9	8'	0.015	0.081	4000	Clayey Sand
B-9	30'	0.022	0.118	4200	Elastic Silt
B-11	10'	0.010	0.165	2200	Silt w/Sand

Consolidation

Boring	Depth	Load	Cv	Load	Cv
B-2	5'	4 ksf	12.4 ft²/day	8 ksf	10.1 ft²/day
B-4	15'	4	2.6	8	12.5
B-9	8'	2	14.6	4	13.4
B-9	30'	4	13.2	8	10.1
B-11	10'	2	13.1	4	9.3

Strength

Undisturbed

B-11 @ 10' Unconfined = 3580 psf (-200 = 79%)

Remolded - Direct Shear

B-2 @ 2' Φ = 35° C = 550 psf remolded to 95% (ASTM D-698?)
B-6 @ 15' 29° 75 } remolded to insitu
B-10 @ 10' 31° 0 } density

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/9/04 BY [illegible]

SUBJECT Soil Characteristics SHEET 2 OF 6

AGEC Data

Index Properties

		mc	DD	+4	-200	LL	PI
B-1	@ 68	16.3	110.4		56	40	26
B-2	@ 34	12.8	86.6	11	54		
B-3	@ 14	4.7	102.5	0	8		
B-3	@ 29	54.7	66.9		87	43	19
	@ 34	20.6	106.5		56	21	7
B-4	@ 14	8.6	89.7	0	50		
	@ 19	9.3	84.0	0	66		
	@ 24	8.9	91.2				

Compression

Boring	Depth	Cr'	Cc'	mpp	Other	Desc.
B-1	68	0.01	0.081		1.0% C.	Sandy Lean Clay
B-3	29	0.008	0.101	2000	–	Lean clay
B-4	19		0.070		5.2%	Collapse Sandy Silt

Strength

Direct Shear

B-2 @ 34	$\phi = 35°$	c = 40 psf	a little gravel
B-3 @ 14	$\phi = 33°$	c = 0	
B-4 @ 14	$\phi = 30°$	c = 100 psf	Silt + Sand

Triaxial Shear

B-4 @ 24	$\phi' = 32°$	c' = 80 psf
	$\phi = 26$	c = 160 psf

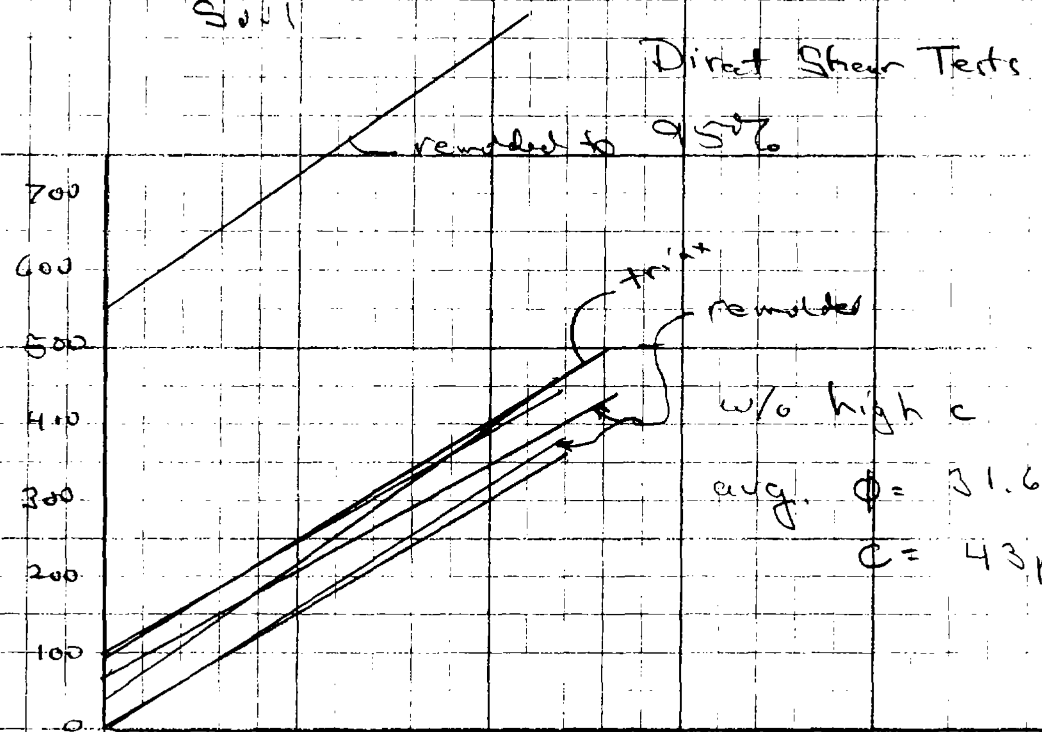
Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/9/04 BY [illegible]

SUBJECT Soil Characteristics SHEET 3 OF 6

Fine Grained Soil

Direct Shear Tests



w/o high c

avg. $\phi = 31.6°$

$c = 43$ psf

Granular Soil

$N_{1(60)}$ avg - 35 Kleinfelder
49 AGEC (we were up the hill further)

Using 35 ⇒ $\phi = 37°$

Comparing the remolded strengths (35 & 550) with the literature -

use $\phi = 32°$ & $c = 300$ psf.

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/9/04 BY [illegible]

SUBJECT Soil Characteristics SHEET 4 OF 6

Unit Weights

Kleinfelder

(SM/SC)		(CL/ML)		(GM)	
15%	87 pcf	41%	77 pcf	1%	132 pcf
27	97	17	88		
16	112	15	87		
7	82	11	96		133 pcf
12	94	46	72		
7	100				
21	107	avg.	103.5		
16	117				
7	96				
20	105				
2	103				
2	106				
15	106				
18	112				
avg.	115 pcf				

AGEC

5	103	16	110
108.2		13	87
		55	67
		21	107
		9	90
		9	84
		9	91
		avg.	106.9

overall

$$\frac{(115)(14) + 108}{15}$$

114.5 pcf

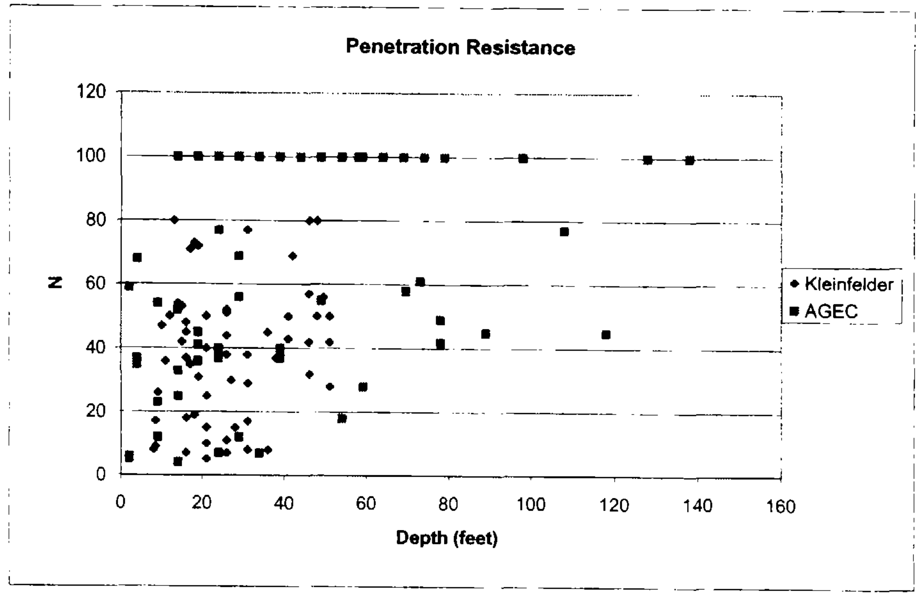
$$\frac{(103.5)(5) + (106.9)(7)}{12}$$

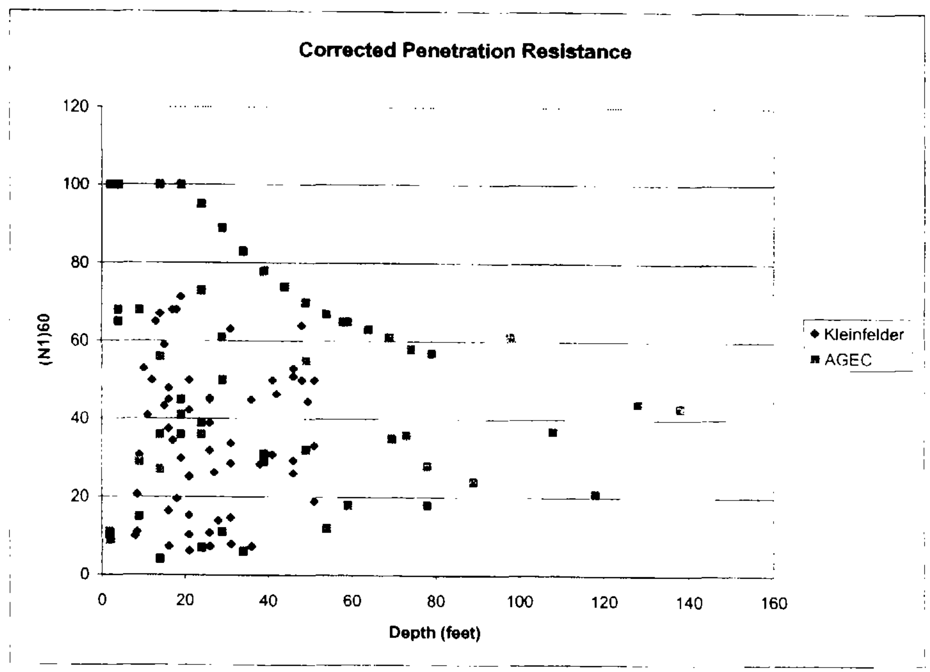
105.5 pcf

1040644

5/6

	Kleinfelder				AGEC		
Boring	Depth	N (raw)	(N1)60	Boring	Depth	N (raw)	(N1)60
1	8.5	17	20.6	1	2	59	100
	16	37	37.5		4	37	68
	21	40	42.2		9	23	29
	26	7	7.3		14	100	100
	51	50	50		19	36	36
2	8.5	9	11		24	100	95
	18	19	19.5		29	69	61
	49.5	56	44.5		34	100	83
3	21	25	25.3		39	100	78
	26	11	10.8		49	55	32
	31	8	7.9		58	100	65
	46	32	26		78	49	28
4	21	50	50		89	45	24
	26	44	38.9		98	100	61
	31	38	33.7		108	77	37
5	16	7	7.3		118	45	21
	21	10	10.3		128	100	44
	36	45	44.9		138	100	43
	41	50	50	2	4	68	100
	46	42	29.3		9	54	68
6	21	5	6.2		14	52	56
	27	30	26.2		19	100	100
	31	77	63.1		24	77	73
	42	69	46.4		29	56	50
7	8	8	9.9		34	7	6
	16	18	16.4		39	37	29
	21	15	15.3		44	100	74
	26	38	31.8		49	100	70
	31	17	14.7		54	18	12
	38	37	28.3		59	100	65
	48	50	50		64	100	63
8	11	36	40.9		69	100	61
	16	45	45		74	100	58
	21	25	25		79	100	57
	26	51	44.9	3	14	33	36
9	10	47	53		19	41	41
	12	50	50		24	37	36
	14	54	67	4	2	6	11
	16	48	48		4	35	65
	18	73	68		9	12	15
	28	15	14		14	4	4
	36	8	7.3		24	7	7
	46	57	51		29	100	89
10	9	26	30.7		34	100	83
	13	80	65		39	40	31
	15	42	43.3		44	100	74
	17	71	68		49	100	55
	19	31	29.8		54	100	67
	21	50	50		59	28	18
	31	29	28.5		64	100	63
	48	80	64		69.5	58	35
	51	42	33.1		73	61	36
11	15	53	59		78	42	18
	17	35	34.4	5	2	5	9
	19	72	71.4		14	25	27
	26	52	45.3		19	45	45
	41	43	30.8		24	40	39
	46	80	53.1		29	12	11
	51	28	18.9				





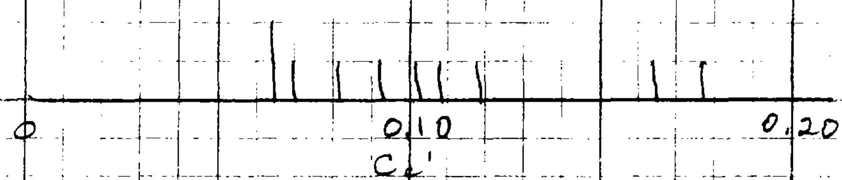
Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/9/04 BY [illegible]

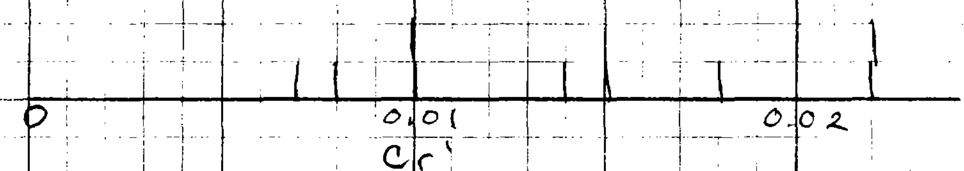
SUBJECT Soil Characteristics SHEET 6 OF 6

Characteristics for analysis

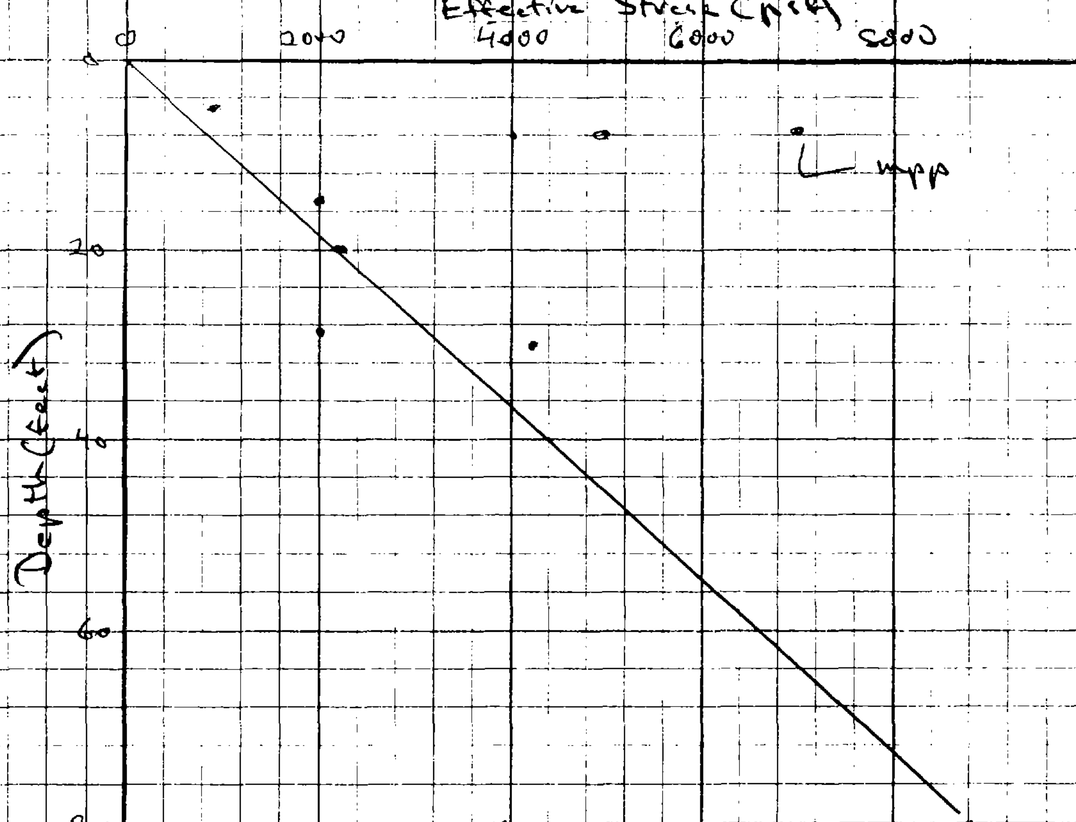
Avg γ clay-silt 107.0



Avg. 0.104



Avg. 0.014



Appears that there is no maximum past pressure

APPENDIX 2

Bearing Capacity

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/17/04 BY SJ

SUBJECT Bearing Capacity SHEET 1 OF 2

A Load applied - Embankment

Embankment $(4265 - 4244) = 21$ ft high

Load is $(21 ft)(105 pcf) = 2205$ psf

Bearing Capacity

Undrained $= \left(\frac{3580 psf}{2}\right)(5.12) = 9165$ psf

$$S.F. = \frac{9165 psf}{2205 psf} = 4.2 \quad \underline{\underline{ok}}$$

Total Stress

$\phi = 26^{\circ} \quad c = 160$ psf

$Q_{ult} = 1.2 c N_c + (0.4) \gamma B N_\gamma$

$N_c = 20$
$N_\gamma = 7$

$= (1.2)(160)(20) + (0.4)(105)(7) B$

$= 3840 + 294 B$

if we assume B = 100 ft

$= 33,240$

$$S.F. = \frac{33,240}{2205} = 15 \quad \underline{ok}$$

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/17/04 BY [illegible]

SUBJECT Bearing Capacity SHEET 2 OF 2

B. Long term - land fill

Load $\underline{(260 \text{ ft high})(120 \text{ pcf})} = 31,200$ psf

Bearing Capacity.

$\phi = 32 \quad c = 80$

$N_\gamma = 18 \quad N_c = 30$

$q_{ult} = (1.2)(80 \text{ psf})(30) + (0.4)(105)(18) B$

$= 2880 + 756 B.$

in order for S.F. = 3.0

$(31,200)(3) = 2880 + 756 B$

$B \geq 120$ ft. ok

we will be much under.

APPENDIX 3

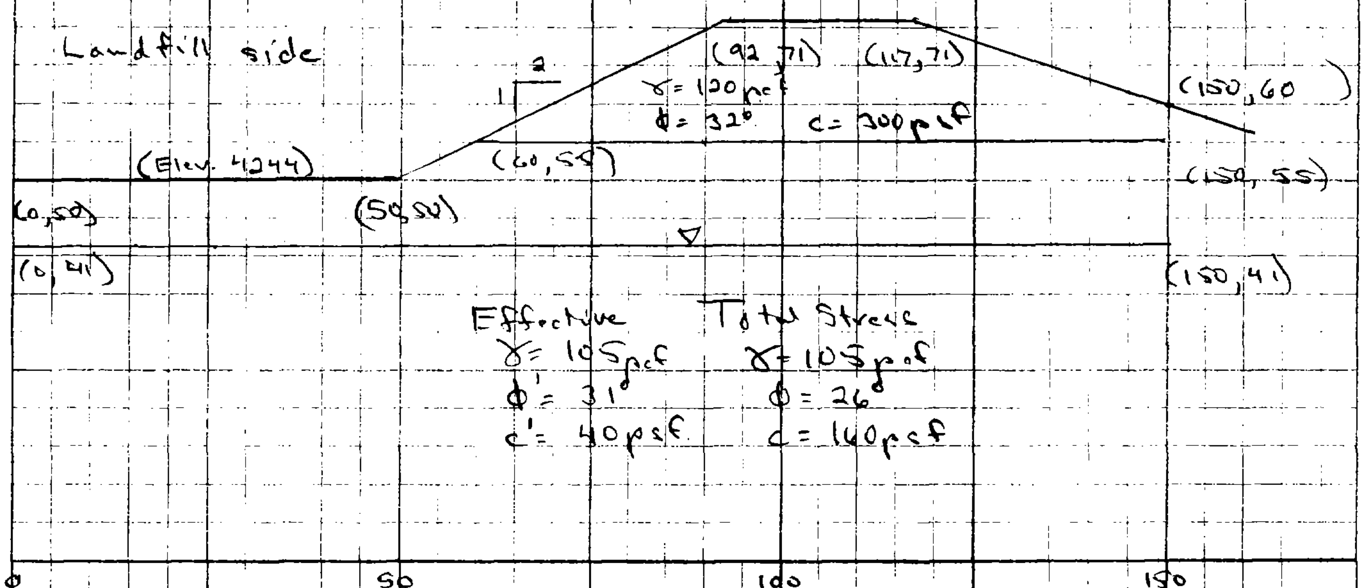
Embankment Stability

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/17/04 BY [illegible]

SUBJECT Emb. Stability SHEET 1 OF 9

Interior Emb Slope

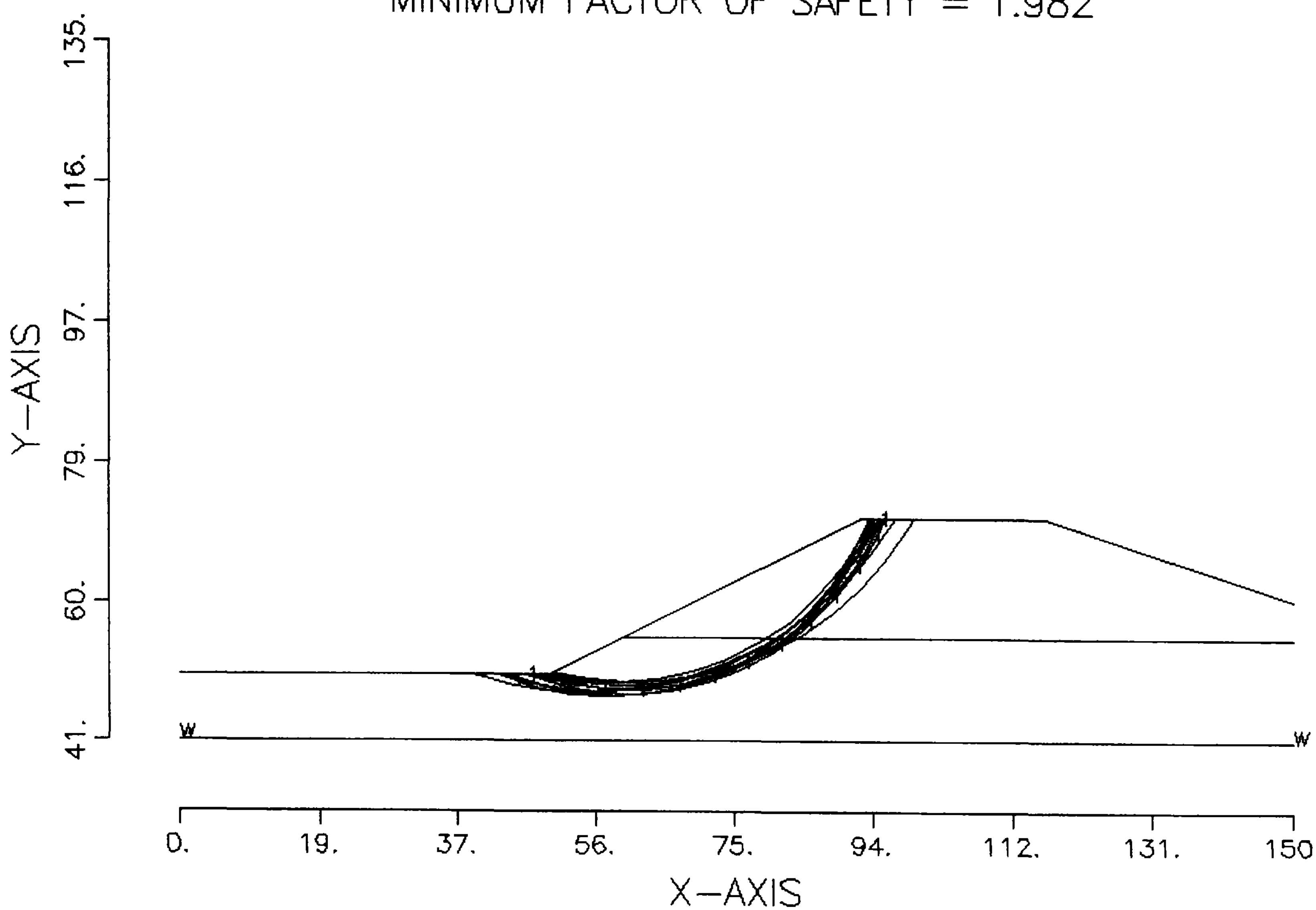


File Input	Output	Condition	S.F
WRL.in1	WRL.ou1	Static	1.982
WRL.in2	WRL.ou2	Seismic	1.625
WRL.in3	WRL.ou3	Static (Total Stress)	2.055

AGL
Midvale UT s/n5206

WRL Embankment Stability – Static

2500SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.982



1040644

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1040644

Problem Title : WRL Embankment Stability - Static
Description :
Remarks :

****
*****   INPUT DATA   *****
****

Profile Boundaries

Number of Boundaries : 6
Number of Top Boundaries : 5

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	50.00	50.00	50.00	2
2	50.00	50.00	60.00	55.00	2
3	60.00	55.00	92.00	71.00	1
4	92.00	71.00	117.00	71.00	1
5	117.00	71.00	150.00	60.00	1
6	60.00	55.00	150.00	55.00	2

Soil Parameters

Number of Soil Types : 2

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	300.0	32.0	0.00	0.0	1
2	105.0	105.0	40.0	31.0	0.00	0.0	1

Piezometric Surfaces

Number of Surfaces : 1
Unit Weight of Water : 62.40 pcf
Piezometric Surface No. : 1
Number of Coordinate Points : 2

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	41.00
2	150.00	41.00

1040644

****

*****   TRIAL SURFACE GENERATION   *****

****

Data for Generating Circular Surfaces

Number of Initiation Points : 50
Number of Surfaces From Each Point : 50
Left Initiation Point : 10.00 ft
Right Initiation Point : 55.00 ft
Left Termination Point : 90.00 ft
Right Termination Point : 140.00 ft
Minimum Elevation : 1.00 ft
Segment Length : 5.00 ft
Positive Angle Limit : 0.00 deg
Negative Angle Limit : 0.00 deg

****

*****   RESULTS   *****

****

Surface No. : 1
Factor of Safety : 1.982
Circle Center X : 60.18 ft
Circle Center Y : 86.31 ft
Circle Radius : 38.41 ft

Slice	X (ft)	Y (ft)	Width (ft)	Weight (lbs)	Load (lbs)	Water (lbs)	Normal (lbs)	Shear (lbs)
1	48.83	49.68	2.35	79.1	0.0	0.0	104.0	80.6
2	51.24	49.02	2.48	415.9	0.0	0.0	485.7	199.1
3	54.95	48.34	4.95	2151.3	0.0	0.0	2280.6	792.5
4	58.71	47.99	2.57	1718.1	0.0	0.0	1721.8	574.0
5	61.21	47.98	2.43	1969.2	0.0	0.0	1973.4	647.4
6	64.91	48.28	4.96	4964.9	0.0	0.0	4809.1	1559.2
7	69.81	49.21	4.84	5789.2	0.0	0.0	5520.3	1774.8
8	74.55	50.78	4.64	6100.0	0.0	0.0	5824.9	1867.2
9	79.04	52.95	4.35	5910.4	0.0	0.0	5747.4	1843.7
10	81.76	54.59	1.09	1473.4	0.0	0.0	1484.4	477.7
11	83.76	56.09	2.90	3758.9	0.0	0.0	3467.4	1643.5
12	87.00	58.93	3.57	4099.4	0.0	0.0	3818.0	1960.9
13	90.33	62.65	3.09	2782.8	0.0	0.0	2527.5	1554.0
14	91.94	64.73	0.13	96.4	0.0	0.0	81.3	63.8
15	93.21	66.88	2.42	1197.7	0.0	0.0	741.7	952.7
16	94.87	69.96	0.89	111.3	0.0	0.0	-298.1	248.8

1040644

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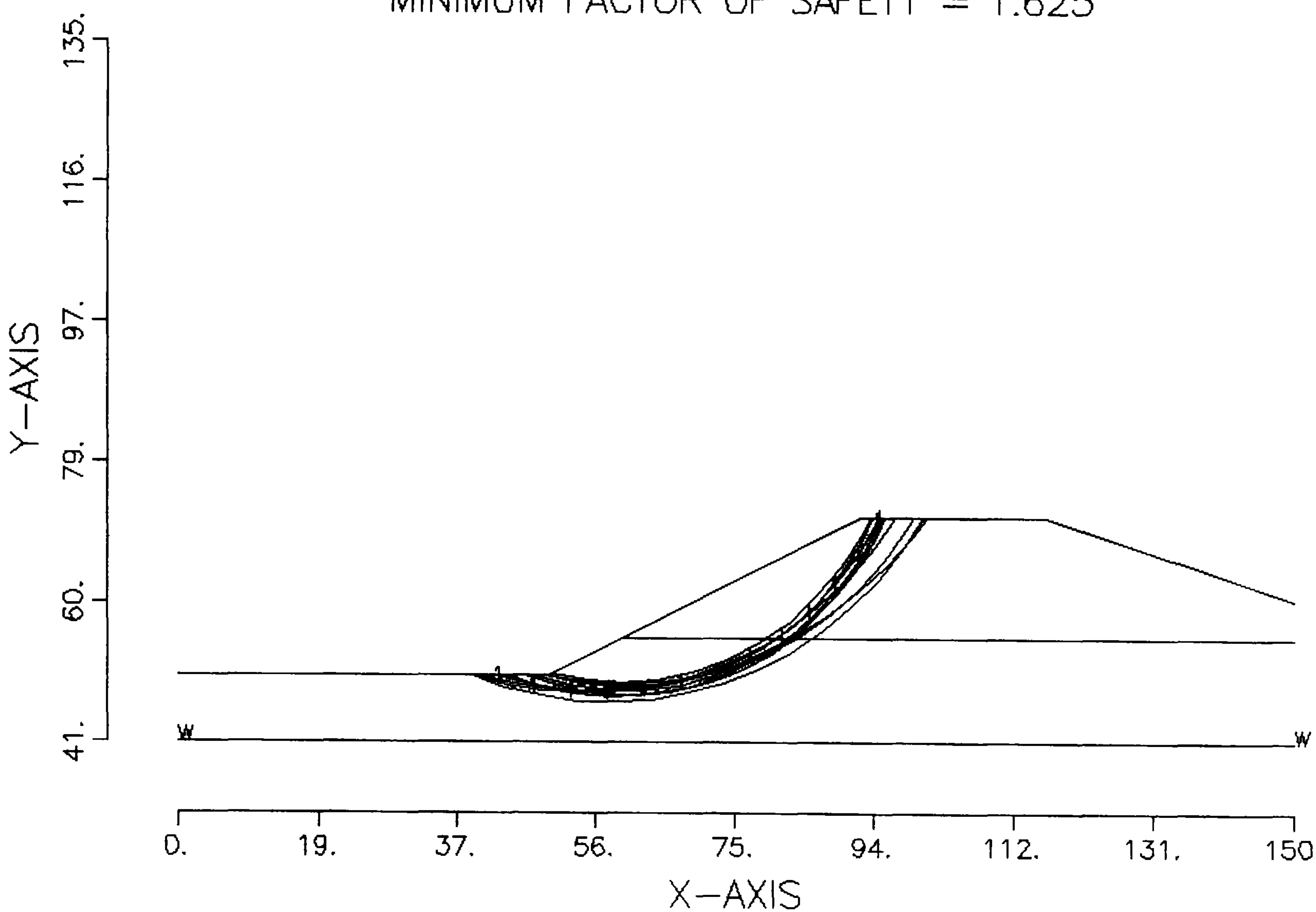
AG

Midvale UT s/n5206

WRL Embankment Stability - Seismic

2500SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.625



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*****                 GeoSlope                  *****
*****               Version 5.00                *****
*****                                      *****
*****          (c)1992 by GEOCOMP Corp, Concord, MA          *****
*****               Licensed to AGEC               *****
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Problem Title : WRL Embankment Stability - Seismic
Description :
Remarks :

```
*****************************************************************************
*****                 INPUT DATA                *****
*****************************************************************************
```

Profile Boundaries

Number of Boundaries : 6
Number of Top Boundaries : 5

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	50.00	50.00	50.00	2
2	50.00	50.00	60.00	55.00	2
3	60.00	55.00	92.00	71.00	1
4	92.00	71.00	117.00	71.00	1
5	117.00	71.00	150.00	60.00	1
6	60.00	55.00	150.00	55.00	2

Soil Parameters

Number of Soil Types : 2

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	300.0	32.0	0.00	0.0	1
2	105.0	105.0	40.0	31.0	0.00	0.0	1

Piezometric Surfaces

Number of Surfaces : 1
Unit Weight of Water : 62.40 pcf
Piezometric Surface No. : 1
Number of Coordinate Points : 2

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	41.00
2	150.00	41.00

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Earthquake Loading

Horizontal Acceleration Coefficient : 0.093

Vertical Acceleration Coefficient : 0.000

****

***** TRIAL SURFACE GENERATION *****

****

Data for Generating Circular Surfaces

Number of Initiation Points : 50

Number of Surfaces From Each Point : 50

Left Initiation Point : 10.00 ft

Right Initiation Point : 55.00 ft

Left Termination Point : 90.00 ft

Right Termination Point : 140.00 ft

Minimum Elevation : 1.00 ft

Segment Length : 5.00 ft

Positive Angle Limit : 0.00 deg

Negative Angle Limit : 0.00 deg

****

***** RESULTS *****

****

Surface No. : 1

Factor of Safety : 1.625

Circle Center X : 56.90 ft

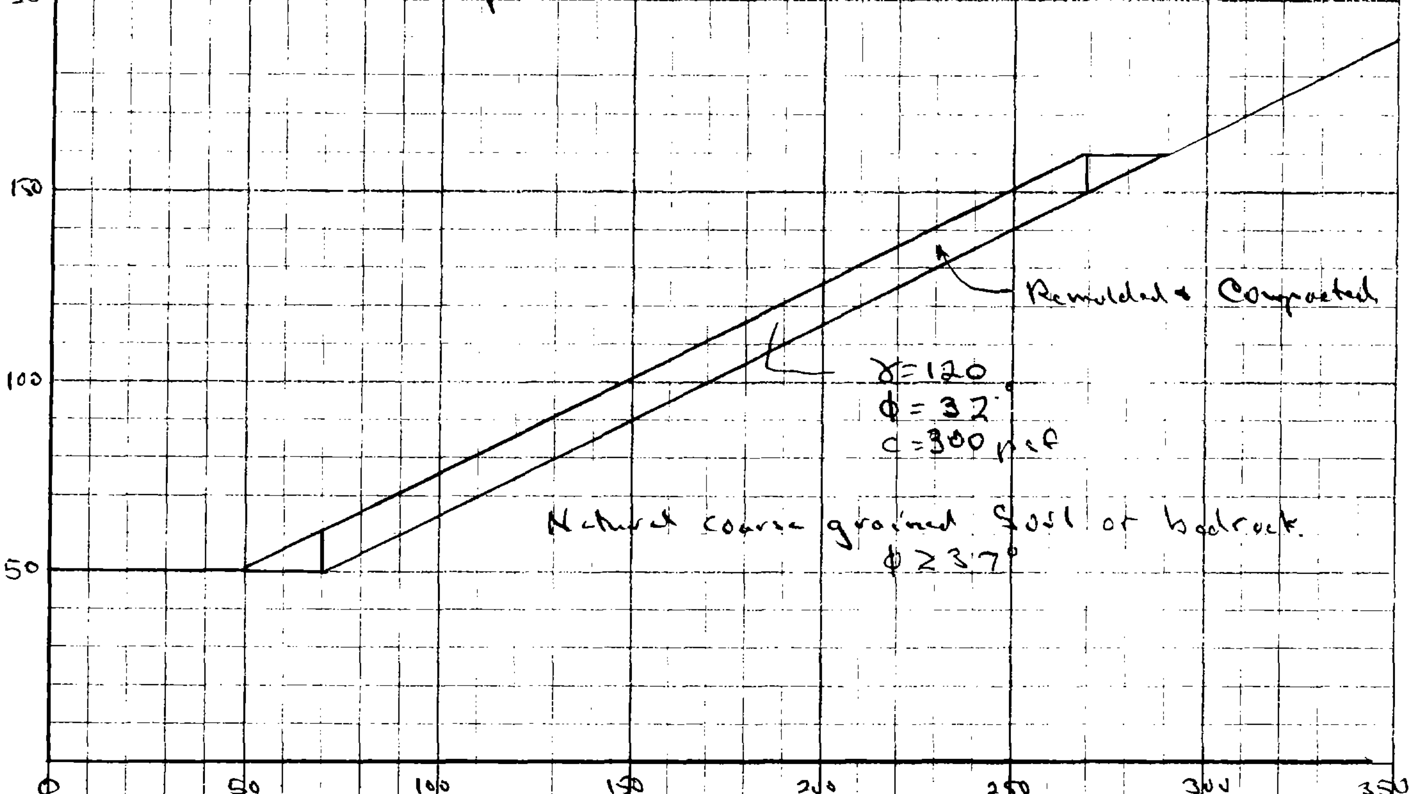
Circle Center Y : 89.77 ft

Circle Radius : 42.11 ft

Slice	X	Y	Width	Weight	Load	Water	Normal	Shear
	(ft)	(ft)	(ft)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)
1	45.47	49.32	4.81	343.6	0.0	0.0	437.6	284.9
2	48.94	48.47	2.13	341.3	0.0	0.0	375.9	192.0
3	51.41	48.08	2.81	774.0	0.0	0.0	844.0	382.2
4	55.31	47.76	5.00	2565.7	0.0	0.0	2608.9	1087.8
5	58.90	47.76	2.19	1540.5	0.0	0.0	1496.3	607.4
6	61.40	47.96	2.79	2297.5	0.0	0.0	2232.5	894.5
7	65.24	48.57	4.90	4850.4	0.0	0.0	4581.0	1817.0
8	70.07	49.85	4.75	5436.8	0.0	0.0	5066.4	1996.5
9	74.71	51.69	4.53	5570.9	0.0	0.0	5194.2	2043.8
10	78.78	53.87	3.62	4510.2	0.0	0.0	4265.0	1682.1
11	80.91	55.19	0.62	768.4	0.0	0.0	662.1	390.3
12	83.17	56.95	3.90	4513.3	0.0	0.0	3855.4	2405.7
13	86.87	60.29	3.51	3425.6	0.0	0.0	2837.3	2014.2
14	90.16	64.05	3.06	2210.9	0.0	0.0	1616.5	1544.8
15	91.84	66.29	0.31	174.6	0.0	0.0	92.0	148.4
16	93.13	68.44	2.25	692.4	0.0	0.0	-3.1	808.9
17	94.41	70.66	0.30	12.3	0.0	0.0	-148.5	80.0

2:1 east facing slope

+ excavation face will be covered with compacted fill.



Slope Stability

Slice	W	α	l	ϕ	c	$W\cos\alpha\tan\phi$	cl	$W\sin\alpha$
1	(1/2)(20)(10)(120) = 12,000	0	20	32	300	7498	6000	0
2	(200)(10)(120) = 240,000	26.56	223	32	300	134142	66900	107,312
3	(1/2)(20)(10)(120) = 12,000	26.56	22.3	32	300	6707	6690	5,365
						148347	79590	112,677

$$S.F. = \frac{148,347 + 79590}{112,677} = 2.02 \text{ ok}$$

$$\text{Seismic SF} = \frac{227,937}{112,677 + (0.0925)(264,000)} = 1.66 \text{ ok}$$

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/17/04 BY

SUBJECT Emb. Stability SHEET 9 OF 9

2:1 slope

w/ $\phi = 37°$

Infinite slope - Static

$$S.F. = \frac{\tan 37°}{\tan 26.5°} = 1.51 \quad ok.$$

Infinite Slope - Seismic

$$= \frac{\cos 26.5°}{\sin 26.5° + K \cos 26.5°} \tan 37°$$

$K = 0.0925$

$S.F. = 1.27 \Rightarrow 1.3$ ok

Summary -

The 2:1 interior Slopes

Lowest S.F. is $\phi = 37°$ $c = 0$

Static	1.51	ok
Seismic	1.3	ok

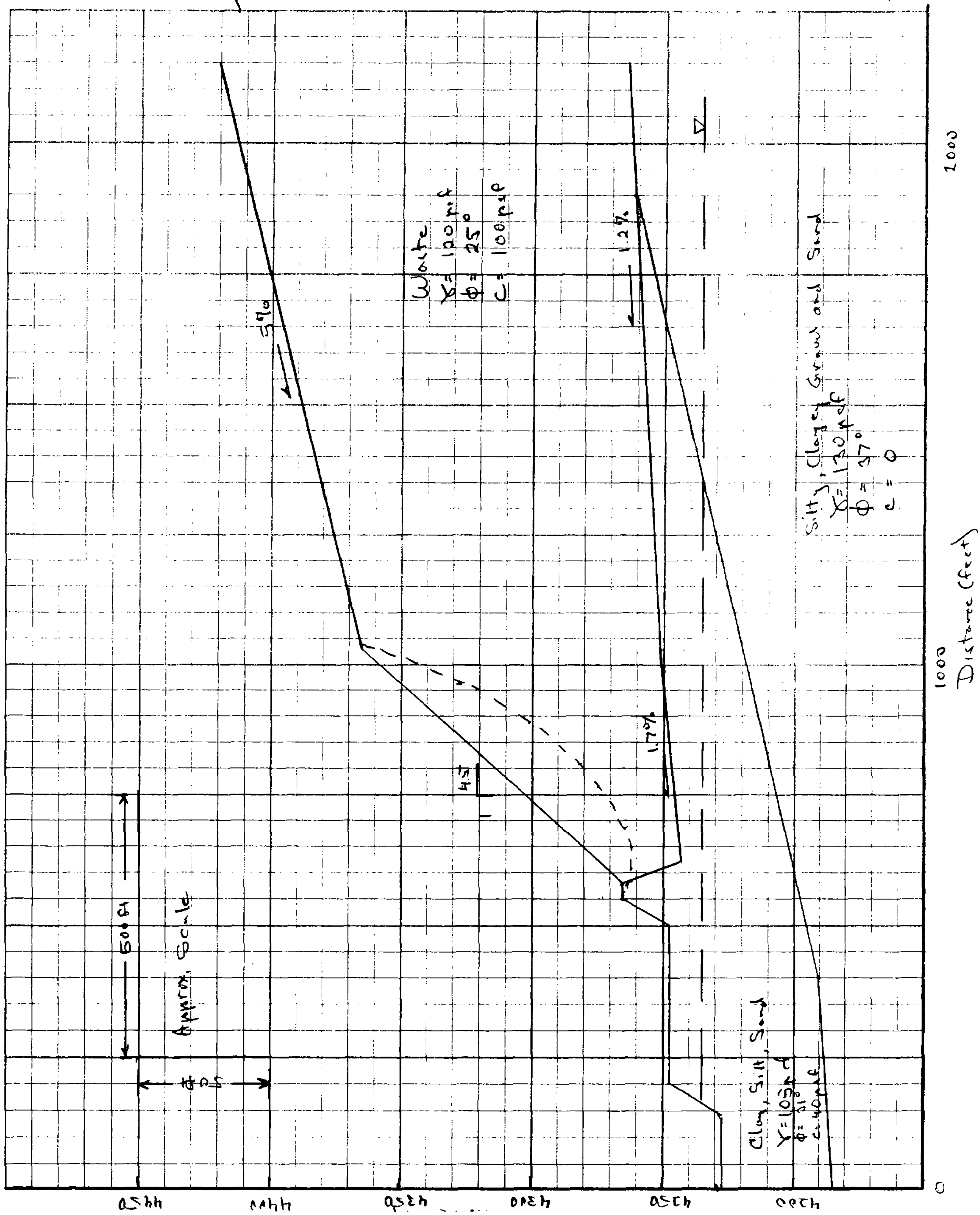
APPENDIX 4

Landfill Stability

Applied Geotechnical Engineering Consultants, P.C.

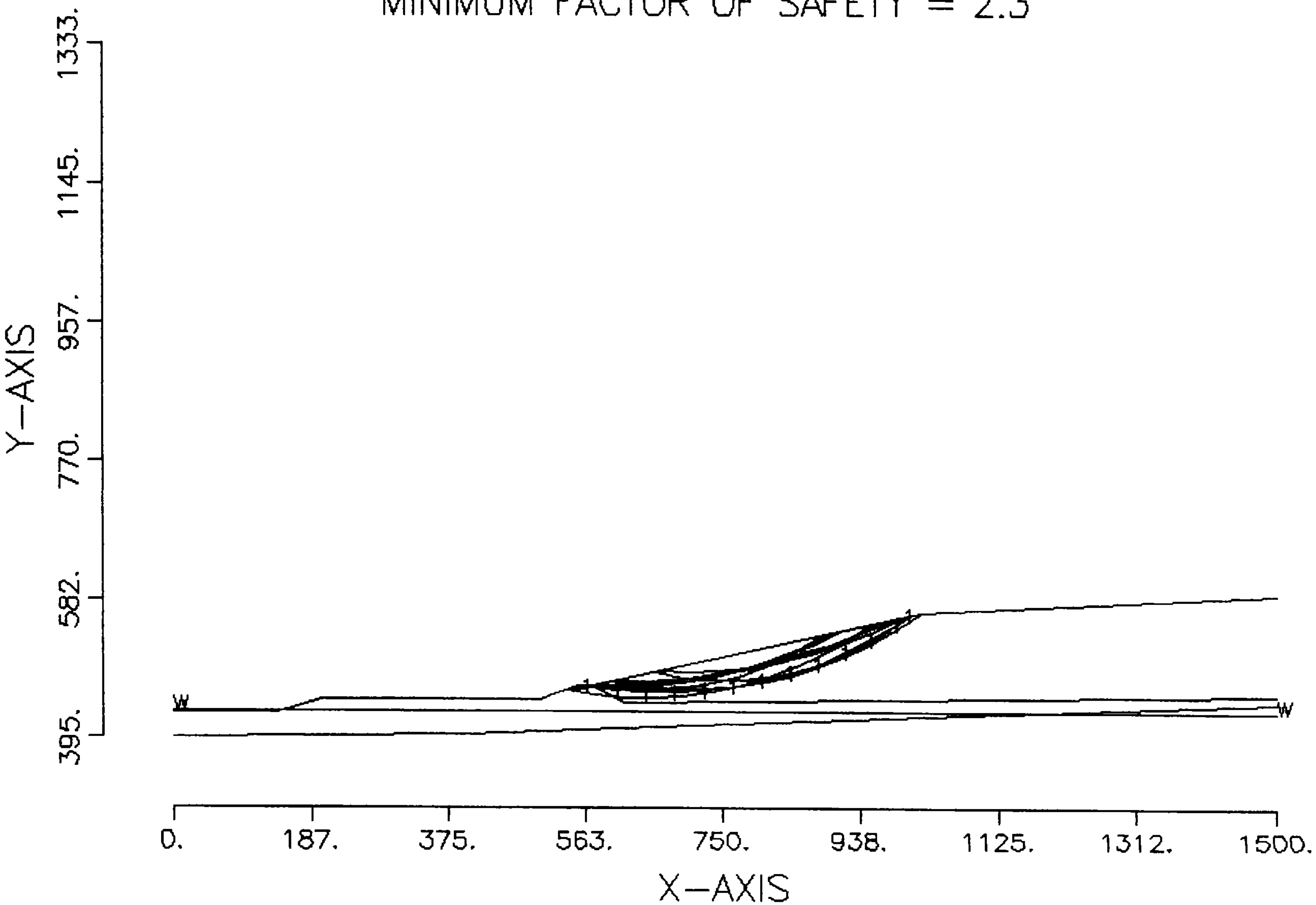
PROJECT NO. 1040644 TITLE WRL DATE 12/17/04 BY [illegible]

SUBJECT Stability SHEET 1 OF 14



AGE
Midvale UT s/n5206

Wasatch Regional Landfill waste slope st
atic
2500SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 2.3



1040644

2/14

1040644

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*****                    GeoSlope                    *****
*****                  Version 5.00                  *****
*****                                                *****
*****       (c)1992 by GEOCOMP Corp, Concord, MA      *****
*****                Licensed to AGEC                *****
******************************************************************************
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Problem Title : Wasatch Regional Landfill waste slope static
Description :
Remarks :

```
******************************************************************************
*****                   INPUT DATA                   *****
******************************************************************************
```

Profile Boundaries

Number of Boundaries : 11

Number of Top Boundaries : 7

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	428.00	140.00	428.00	2
2	140.00	428.00	200.00	448.00	2
3	200.00	448.00	500.00	448.00	2
4	500.00	448.00	551.00	465.00	2
5	551.00	465.00	571.00	465.00	2
6	571.00	465.00	1021.00	565.00	1
7	1021.00	565.00	1500.00	590.00	1
8	571.00	465.00	613.00	444.00	2
9	613.00	444.00	1500.00	453.00	2
10	0.00	395.00	400.00	400.00	3
11	400.00	400.00	1500.00	443.00	3

Soil Parameters

Number of Soil Types : 3

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	100.0	25.0	0.00	0.0	1
2	105.0	105.0	40.0	31.0	0.00	0.0	1
3	130.0	130.0	0.0	37.0	0.00	0.0	1

Piezometric Surfaces

Number of Surfaces : 1

Unit Weight of Water : 62.40 pcf

1040644

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Piezometric Surface No. : 1
Number of Coordinate Points : 2

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	430.00
2	1500.00	430.00

****
*****   TRIAL SURFACE GENERATION   *****
****

Data for Generating Circular Surfaces
Number of Initiation Points : 50
Number of Surfaces From Each Point : 50
Left Initiation Point : 450.00 ft
Right Initiation Point : 800.00 ft
Left Termination Point : 950.00 ft
Right Termination Point : 1400.00 ft
Minimum Elevation : 1.00 ft
Segment Length : 40.00 ft
Positive Angle Limit : 0.00 deg
Negative Angle Limit : 0.00 deg

****
*****   RESULTS   *****
****

Surface No. : 1
Factor of Safety : 2.353
Circle Center X : 621.35 ft
Circle Center Y : 1362.72 ft
Circle Radius : 900.88 ft

Slice	X (ft)	Y (ft)	Width (ft)	Weight (lbs)	Load (lbs)	Water (lbs)	Normal (lbs)	Shear
1	550.50	464.64	1.00	20.5	0.0	0.0	21.8	22.6
2	561.00	464.04	20.00	2019.7	0.0	0.0	2072.7	869.6
3	572.73	463.37	3.46	798.1	0.0	0.0	814.6	266.9
4	582.20	462.83	15.47	8656.0	0.0	0.0	8807.4	2403.6
5	609.93	462.13	40.00	55295.6	0.0	0.0	55461.0	12688.4
6	649.92	462.51	39.98	96078.1	0.0	0.0	95472.2	20616.1
7	689.85	464.67	39.88	128002.4	0.0	0.0	126335.3	26731.2
8	729.65	468.59	39.71	150877.1	0.0	0.0	148217.0	31066.7
9	769.23	474.28	39.46	164631.5	0.0	0.0	161294.2	33657.8
10	808.53	481.72	39.13	169316.0	0.0	0.0	165756.5	34541.9
11	847.45	490.90	38.72	165101.6	0.0	0.0	161808.1	33759.6
12	885.93	501.79	38.24	152277.6	0.0	0.0	149670.4	31354.7
13	923.88	514.39	37.68	131249.5	0.0	0.0	129583.3	27374.7
14	961.24	528.65	37.04	102534.6	0.0	0.0	101808.5	21871.5
15	997.93	544.56	36.34	66757.6	0.0	0.0	66630.7	14901.6
16	1018.55	554.19	4.90	6039.2	0.0	0.0	6054.8	1434.0
17	1031.31	560.76	20.62	11815.3	0.0	0.0	11599.4	3284.0

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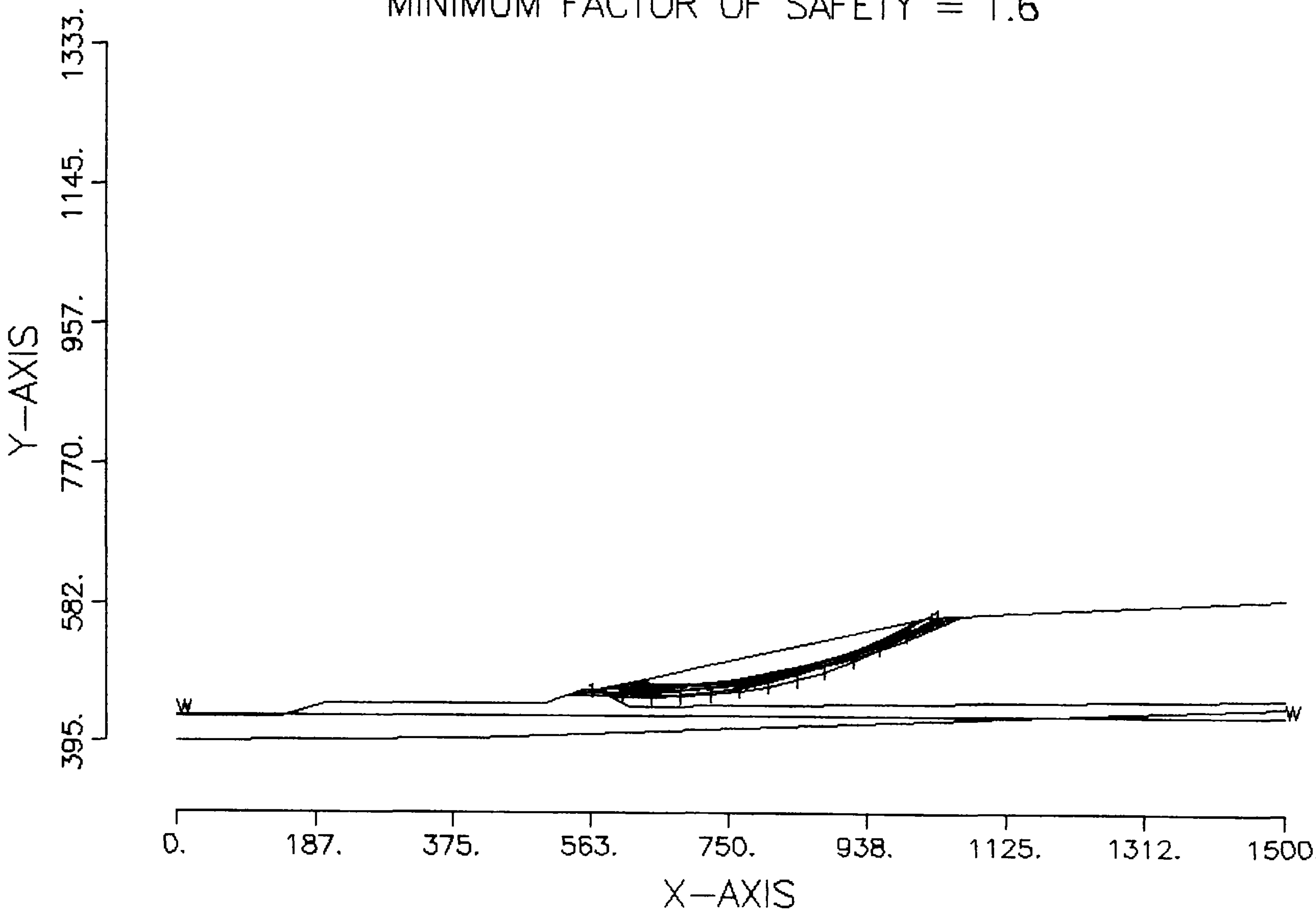
AGE
Midvale UT s/n5206

Wastach Regional Landfill waste slope dy
namic
2500SURFACES HAVE BEEN GENERATED
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 1.6



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```
*************************************************************************************
*****                    GeoSlope                    *****
*****                  Version 5.00                  *****
*****                                                *****
*****     (c)1992 by GEOCOMP Corp, Concord, MA       *****
*****               Licensed to AGEC                 *****
*************************************************************************************
```

Problem Title : Wastach Regional Landfill waste slope dynamic
Description :
Remarks :

```
*************************************************************************************
*****                   INPUT DATA                   *****
*************************************************************************************
```

Profile Boundaries

Number of Boundaries : 11
Number of Top Boundaries : 7

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	428.00	140.00	428.00	2
2	140.00	428.00	200.00	448.00	2
3	200.00	448.00	500.00	448.00	2
4	500.00	448.00	551.00	465.00	2
5	551.00	465.00	571.00	465.00	2
6	571.00	465.00	1021.00	565.00	1
7	1021.00	565.00	1500.00	590.00	1
8	571.00	465.00	613.00	444.00	2
9	613.00	444.00	1500.00	453.00	2
10	0.00	395.00	400.00	400.00	3
11	400.00	400.00	1500.00	443.00	3

Soil Parameters

Number of Soil Types : 3

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	100.0	25.0	0.00	0.0	1
2	105.0	105.0	40.0	31.0	0.00	0.0	1
3	130.0	130.0	0.0	37.0	0.00	0.0	1

Piezometric Surfaces

Number of Surfaces : 1
Unit Weight of Water : 62.40 pcf

Piezometric Surface No. : 1
Number of Coordinate Points : 2

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Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	430.00
2	1500.00	430.00

Earthquake Loading

Horizontal Acceleration Coefficient : 0.093

Vertical Acceleration Coefficient : 0.000

****

*****   TRIAL SURFACE GENERATION   *****

****

Data for Generating Circular Surfaces

Number of Initiation Points : 50

Number of Surfaces From Each Point : 50

Left Initiation Point : 450.00 ft

Right Initiation Point : 800.00 ft

Left Termination Point : 950.00 ft

Right Termination Point : 1400.00 ft

Minimum Elevation : 1.00 ft

Segment Length : 40.00 ft

Positive Angle Limit : 0.00 deg

Negative Angle Limit : 0.00 deg

****

*****   RESULTS   *****

****

Surface No. : 1

Factor of Safety : 1.628

Circle Center X : 621.35 ft

Circle Center Y : 1362.72 ft

Circle Radius : 900.88 ft

Slice	X (ft)	Y (ft)	Width (ft)	Weight (lbs)	Load (lbs)	Water (lbs)	Normal (lbs)	Shear (lbs)
1	550.50	464.64	1.00	20.5	0.0	0.0	22.4	32.9
2	561.00	464.04	20.00	2019.7	0.0	0.0	2095.3	1265.2
3	572.73	463.37	3.46	798.1	0.0	0.0	821.6	388.3
4	582.20	462.83	15.47	8656.0	0.0	0.0	8869.6	3491.7
5	609.93	462.13	40.00	55295.6	0.0	0.0	55532.9	18358.7
6	649.92	462.51	39.98	96078.1	0.0	0.0	95183.5	29713.0
7	689.85	464.67	39.88	128002.4	0.0	0.0	125446.8	38379.2
8	729.65	468.59	39.71	150877.1	0.0	0.0	146597.7	44435.9
9	769.23	474.28	39.46	164631.5	0.0	0.0	158913.2	47962.6
10	808.53	481.72	39.13	169316.0	0.0	0.0	162676.3	49040.2
11	847.45	490.90	38.72	165101.6	0.0	0.0	158179.3	47752.4
12	885.93	501.79	38.24	152277.6	0.0	0.0	145727.0	44186.6
13	923.88	514.39	37.68	131249.5	0.0	0.0	125639.3	38434.3
14	961.24	528.65	37.04	102534.6	0.0	0.0	98254.6	30592.5
15	997.93	544.56	36.34	66757.6	0.0	0.0	63932.4	20764.0
16	1018.55	554.19	4.90	6039.2	0.0	0.0	5768.1	1990.4
17	1031.31	560.76	20.62	11815.3	0.0	0.0	10942.8	4558.2

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SUBJECT ______ SHEET ______ OF ______

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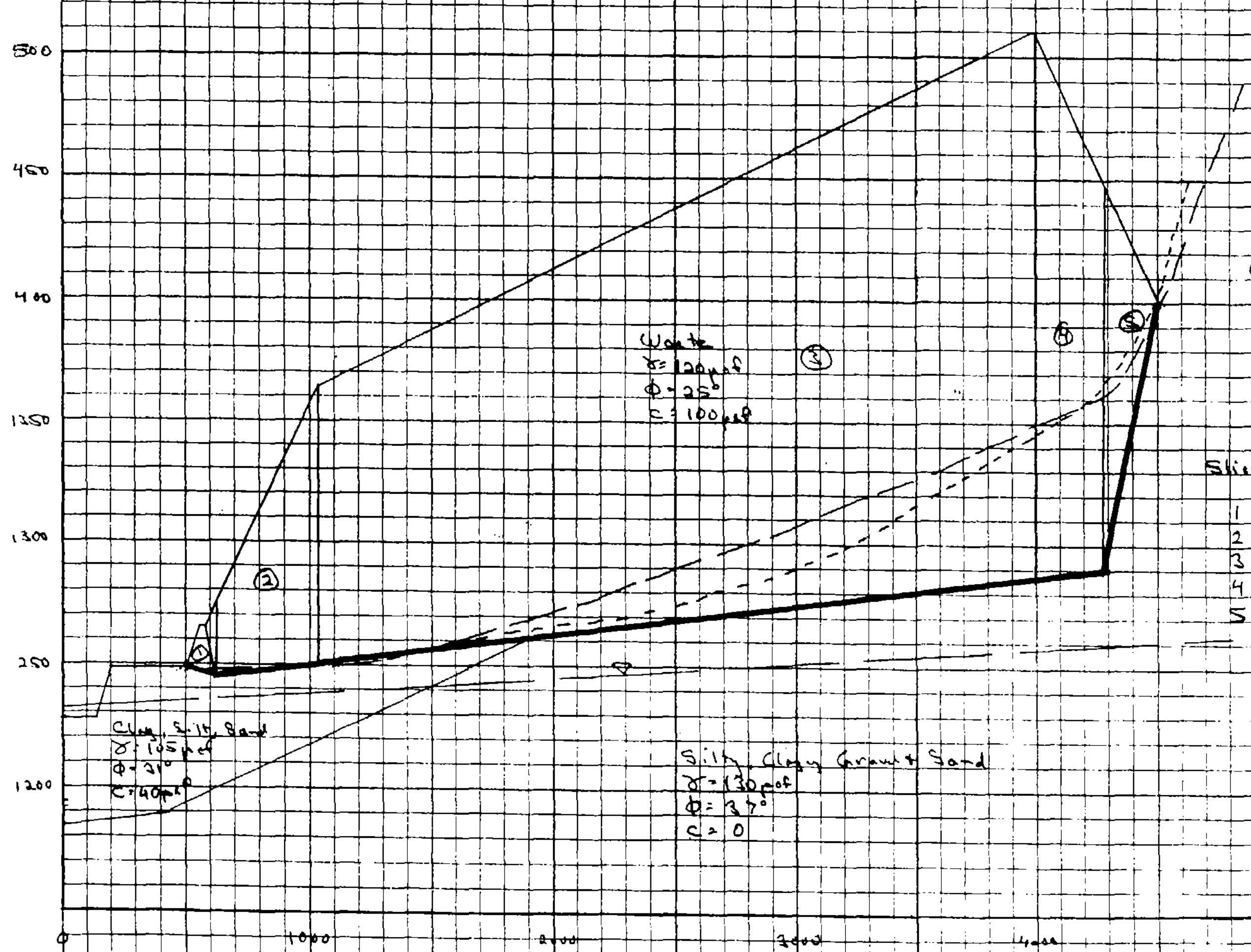
SUBJECT ______ SHEET ______ OF ______



Sliding Wedge Stability

Weight

1	- (130)(200)(105)	=	2,730,000
2	- (410)(30 + 112)(½)(120)	=	3,517,800
3	- (2950)(112 + 224)(½)(120)	=	59,649,000
4	- (280)(224 + 157)(½)(120)	=	6,400,800
5	- (220)(157)(½)(120)	=	2,072,400

Slice	W	l	α	c	φ	W cos α tan φ	cl	W sin α
1	2,730,000	130	-1.94°	40	31	1,639,400	5200	-92,418
2	3,517,800	410	0.974°	0	8	494,323	-	59,798
3	59,649,000	2950	0.688°	0	8	8,383,515	-	716,240
4	6,400,800	280	0.688°	0	8	899,508	-	76,858
5	2,072,400	237	27.6°	95	23.9	813,855	22,515	760,135
						12,229,601	27,715	1,720,613

$$S.F. = \frac{12,229,604 + 27715}{1720613} = 7.1 \quad ok$$

Seismic - Add Eq to driving (W)(Eq)

- (0.2103)(0.44) = 0.0925

$$SF = 12,257,316 / [1720613 + (0.0925)(74,370,000)]$$

1.43 ok

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Basis of Analysis

- Infinite Slope (if ϕ only)
- Seismic $a = 0.21g$ (2% in 50 yrs.)
  reduced $(0.44)(0.21) = 0.0925$
- Safety Factor

$$\text{Static S.F.} = \frac{\tan\phi}{\tan\alpha}$$

$$\text{Seismic S.F.} = \frac{\cos\alpha}{\sin\alpha + K\cos\alpha}\tan\phi$$

- If ϕ + c material.

$$\text{Static S.F.} = \frac{W\cos\alpha\tan\phi + cl}{W\sin\alpha}$$

$$\text{Seismic S.F.} = \frac{W\cos\alpha\tan\phi + cl}{W\sin\alpha + KW}$$

- Calculations

Location	Slope	Strength Friction	Strength Cohesion	S.F. Static	S.F. Seismic
Floor	1.7%	8°	0	8.3	1.3
Interior Side	2:1	18	50	See soil cover section	
		26	30		
		23.9	95		
Exterior Top	5%	~~21.5°~~ 18	~~84~~ 50	* ~~3.8~~ 10.7	~~2.8~~ 3.7
Exterior Side	4:1	~~23.9°~~ 18	~~95~~ 50	* ~~1.8~~ (~~w/o cohesion~~)	~~1.3~~
				2.2	1.6

* See next page

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SUBJECT Geosynthetic Stability SHEET 9a OF 14

- Plan now includes the possibility of a GCL in the closure cap.

- The weakest condition is within the GCL

Static $S.F. = \dfrac{W \cos\alpha \tan\phi + c l}{W \sin\alpha}$

Top $\alpha = 2.86°$
$W = (120\,pcf)(2\,ft)$
$= 240\,psf/ft$

Slope $\alpha = 14.04°$
$W = 240\,psf/ft$

Top
$S.F. = \dfrac{(240)(\cos 2.86)(\tan 18°) + (50\,psf)(1\,ft)}{(240)\sin 2.86}$

$= 10.7$ ok

Slope
$S.F. = \dfrac{(240)(\cos 14.04)(\tan 18°) + (50)(1)}{(240)\sin 14.04}$

$= 2.16$ ok

Seismic
Top
$S.F. = \dfrac{(240)\cos 2.86 \tan 18 + (50)(1)}{(240)\sin 2.86 + (0.0925)(240)}$

$= 3.7$ ok

Side Slope

$S.F. = \dfrac{(240)(\cos 14.04)(\tan 18) + (50)(1)}{(240)\sin 14.04 + (0.0925)(240)}$

$= 1.56$ ok

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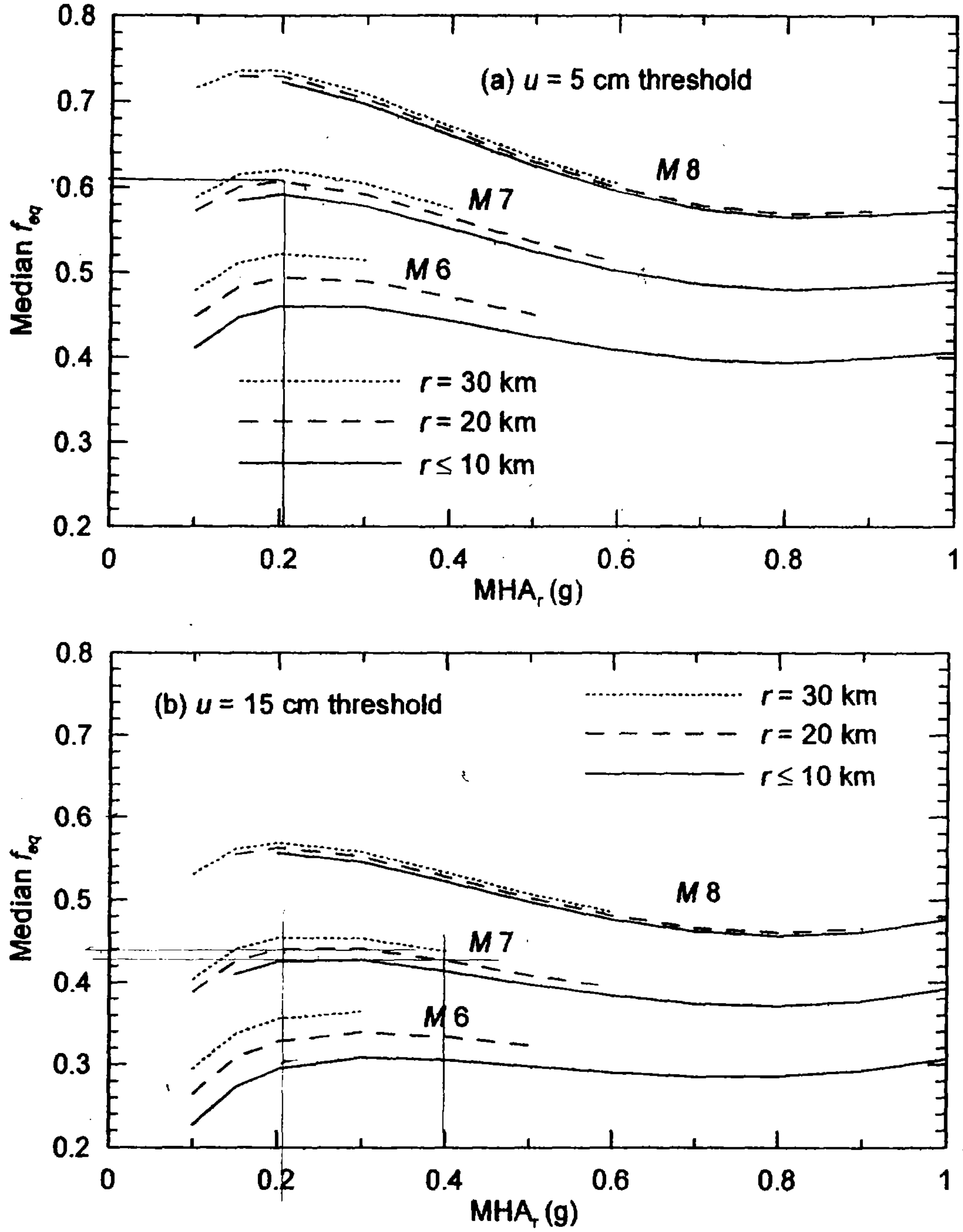

Figure 11.1. Required Values of f_{eq} as Function of MHA$_r$ and Seismological Condition for Threshold Displacements of (a) 5 cm and (b) 15 cm

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SUBJECT Stability SHEET 11 OF 14

Floor Stability

$$S.F. = \frac{\tan 8^\circ}{\tan 0.974} = 8.3 \text{ ok}$$

$$S.F._{eq} = \frac{(\cos 0.974)(\tan 8^\circ)}{(\sin 0.974) + (0.0925 \cos 0.974)} = 1.28 > 1.3 \text{ ok}$$

Interior Side

Assume 47' Soil Cover (2:1) $w = (2)(47)(120 \text{ pcf})$
$= 11,280$ plf

$$\text{Static } SF = \frac{(11280) \cos 26.5 \tan \phi + c(47)}{(11280) \sin 26.5}$$

$$= \frac{10,095 \tan \phi + 47c}{5033}$$

ϕ	c	S.F.	
18°	50 psf	1.12	(hydrated bentonite)
26°	30 psf	1.26	(GCL/Soil)
23.9°	95 psf	1.78	(Soil cover/HDPE)

Need to consider flattening cover slope and or include the passive (toe) resistance.

See Appendix 5 for soil cover stability

AGEC

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SUBJECT Stability SHEET 12 OF 14

Closure Top.

5% $\phi = 21.5$ $c = 84$ psf

Using friction only

Static S.F. $= \dfrac{\tan 21.5}{\tan 2.86} = \underline{\underline{7.9 \text{ ok}}}$

Seismic S.F. $= \dfrac{\cos 2.86 \tan 21.5}{\sin 2.86 + (0.0925)\cos 2.86}$

$= \underline{\underline{2.76 \text{ ok}}}$

with seepage down slope (full)

$r_u = \dfrac{\gamma_w}{\gamma} \cos^2 \alpha$

$= \dfrac{62.4}{120} \cos^2 2.86 = 0.52$

$A = 0.48$

S.F. $= (0.48) \dfrac{\tan 21.5}{\tan 2.86} = \underline{\underline{3.78 \text{ ok}}}$

1040644·

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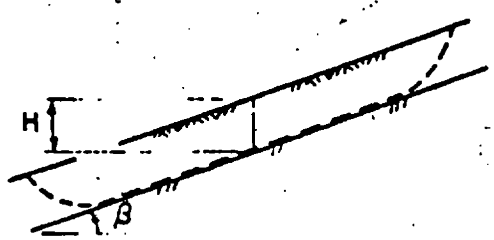


γ = total unit weight of soil

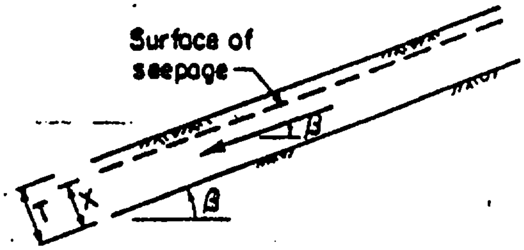
γ_w = unit weight of water

c' = cohesion intercept } Effective Stress

ϕ' = friction angle } Effective Stress

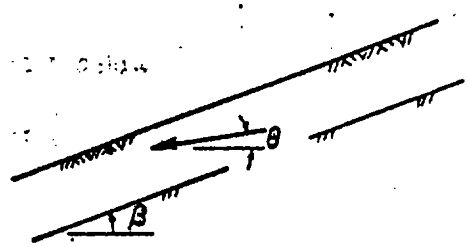
r_u = pore pressure ratio = $\frac{u}{\gamma H}$

u = pore pressure at depth H



Seepage parallel to slope

$$r_u = \frac{X}{T}\,\frac{\gamma_w}{\gamma}\cos^2\beta$$



Seepage emerging from slope

$$r_u = \frac{\gamma_w}{\gamma}\,\frac{1}{1+\tan\beta\tan\theta}$$

Steps:

1. Determine r_u from measured pore pressures or formulas at right
2. Determine A and B from charts below
3. Calculate $F = A\,\frac{\tan\phi'}{\tan\beta} + B\,\frac{c'}{\gamma H}$

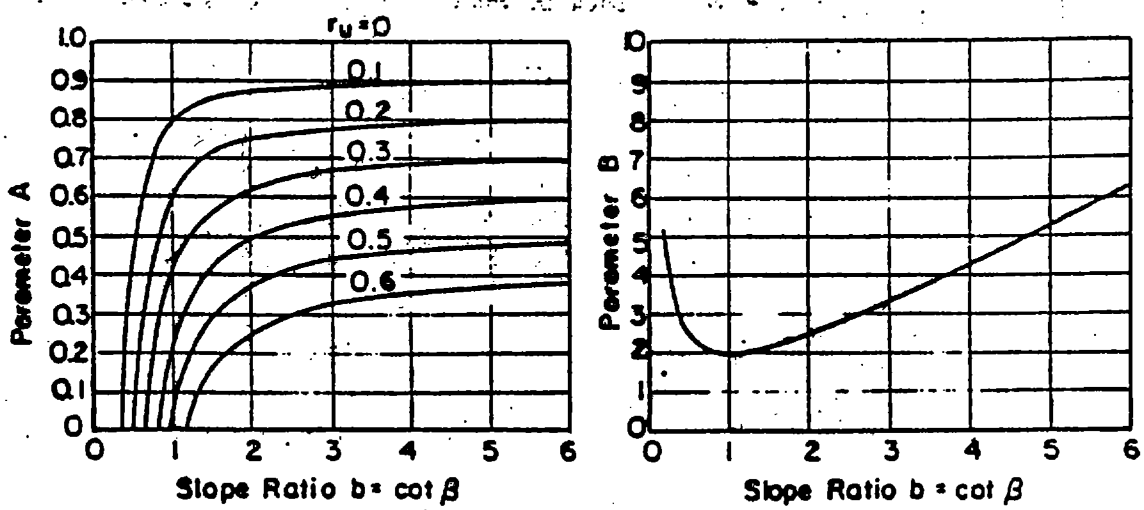


Fig. 10 STABILITY CHARTS FOR INFINITE SLOPES.

Fig. 11 SLOPE STABILITY CHARTS FOR φ = 0, AND STRENGTH INCREASING WITH DEPTH. (after Hunter and Schuster, ...)

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SUBJECT Stability SHEET 14 OF 14

4:1 Side Slopes

$\phi = 23.9°$ $c = 95$ psf

Static S.F. = $\frac{\tan 23.9°}{\tan 14.0°} = \underline{\underline{1.8}}$ ok

Seismic S.F. = $\frac{\cos 14.0 \tan 23.9°}{\sin 14.0 + (0.0925) \cos 14.0} = \underline{\underline{1.3}}$ ok

APPENDIX 5

Soil Cover Stability

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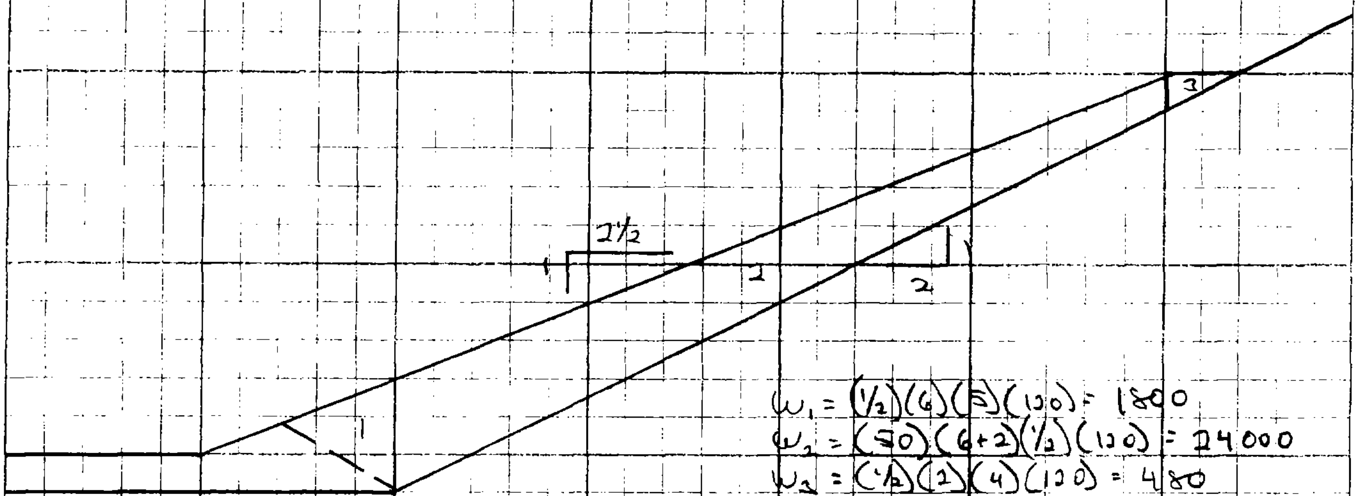
SUBJECT Cover Stability (Protective) SHEET 1 OF 6



$W_1 = (1/2)(6)(5)(120) = 1800$

$W_2 = (50)(6+2)(1/2)(120) = 24000$

$W_3 = (1/2)(2)(4)(120) = 480$

Slice	W	α	l	φ	c	W cos α tan φ	c l	W sin α
1	1800	-30.9	7	25	100	720	700	-924
2	24000	26.5	44.7	18	50	6979	2235	10,709
3	480	26.5	4.5	18	50	140	225	214
						7839	3160	9999

$S.F. = \frac{7839 + 3160}{9999} = 1.10$

(harsh leachate)

Slice								
1	1800	-30.9	7	25	100	720	700	-924
2	24000	26.5	44.7	26	30	10,476	1341	10,709
3	480	26.5	4.5	26	30	210	135	214
						11,406	2176	9999

$S.F. = \frac{11,406 + 2176}{9990} = 1.36$

(GCL/Soil)

add 50% tension of fabric in GCL 360 lb/ft

S.F. = 1.4

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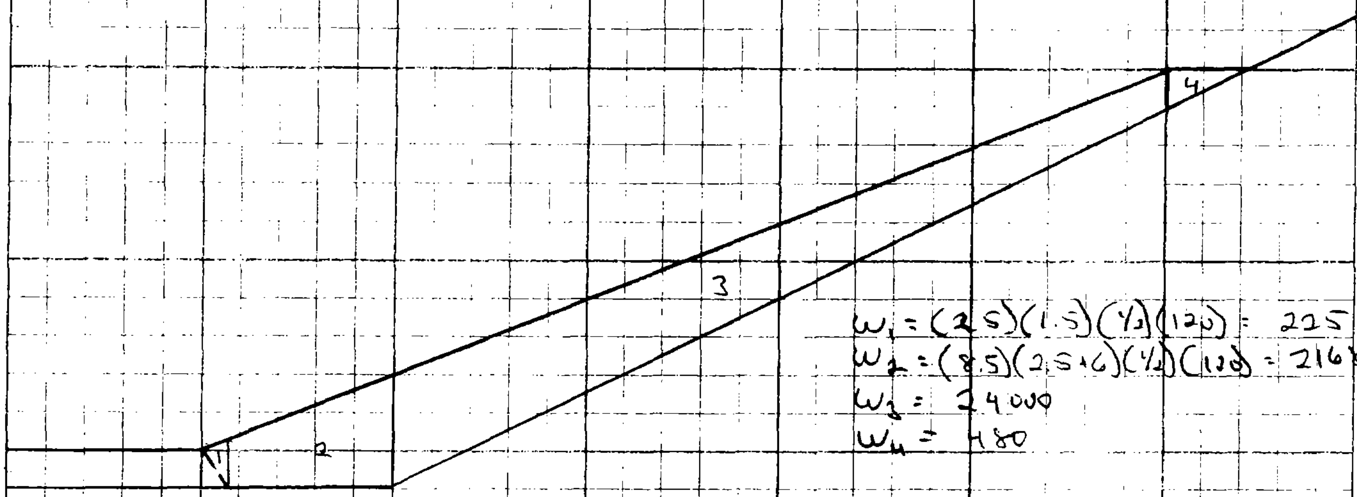
SUBJECT PC Stability SHEET 2 OF 6



$W_1 = (2.5)(1.5)(1/2)(120) = 225$

$W_2 = (8.5)(2.5+6)(1/2)(120) = 2168$

$W_3 = 24000$

$W_4 = 480$

Slice	W	α	l	φ	c	W cos α tan φ	cl	W sin α
1	225	-57.5	2	25	100	56	200	-190
2	2168	0	8.5	8	0	305	0	0
3	24000	26.5	44.7	26	30	10475	1341	10,709
4	480	26.5	4.5	26	30	210	135	214
						11,046	1676	10,733

$$S.F. = \frac{11,047 + 1676}{10,733} = 1.2$$

+ tension - 360 lb/ft

S.F. = 1.2

Conclusion - The interface between synthetic materials and synthetic/soil should be verified.

- The S.F. on the order of 1.2 to 1.4 (as calculated) will be higher when actual laboratory tests indicate higher shear strengths. In other words - the strengths used in this analysis are low!

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SUBJECT PC Stability SHEET 3 OF 6

Seismic

$$S.F. = \frac{11047 + 1676 + 360}{10{,}733 + (0.0925)(26{,}873)} = 0.99$$

1.0

What tension is needed for S.F. = 1.5

$$1.5 = \frac{11{,}047 + 1676 + T}{10{,}733}$$

$$T = 3376$$

What friction below GCL (cohesion too)

$$1.5 = \frac{361 + 210 + 1676 + \frac{10685}{\tan 26}(\tan\phi) + 200 + \frac{1476}{30}(c)}{10{,}733}$$

$$13652 = 21907 \tan\phi + 49.2\,c$$

ϕ	c
29.0	30
26.0	60

(Much of the literature indicates that these values are realistic).

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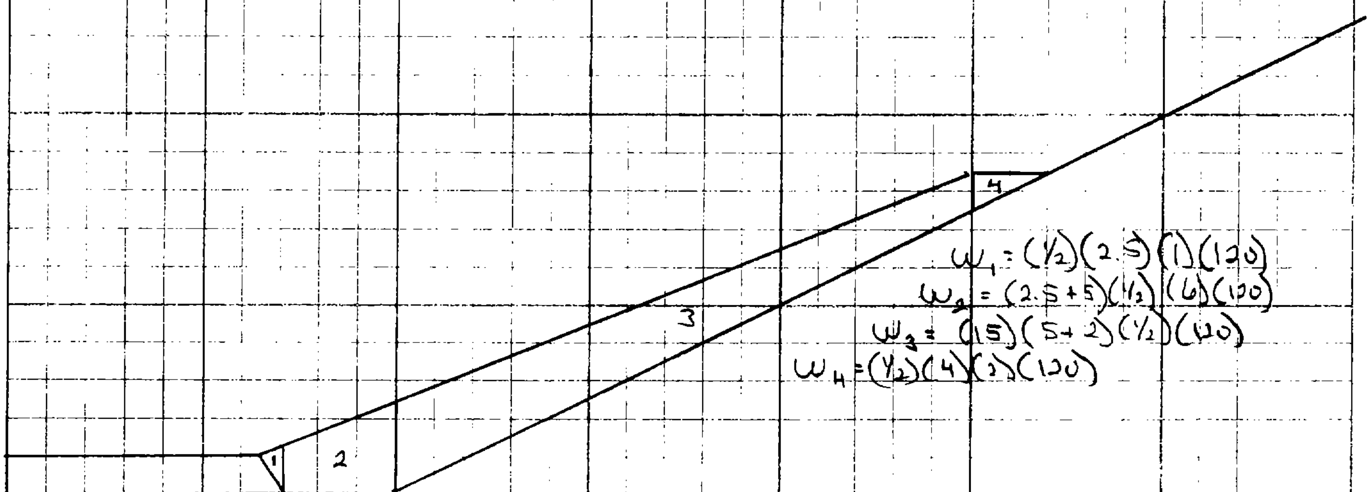
SUBJECT PC Stability SHEET 4 OF 6

How high can we go and maintain S.F. = 1.5

Try 15' high



$W_1 = (1/2)(2.5)(1)(120)$

$W_2 = (2.5+5)(1/2)(6)(120)$

$W_3 = (15)(5+2)(1/2)(120)$

$W_4 = (1/2)(4)(2)(120)$

Slice	W	α	l	φ	c	W cos α tan φ	c l	W sin α
1	150	-57.5	2	25	100	38	200	-127
2	2700	0	6	8	0	379	0	0
3	13,889	26.5	33.5	26	30	6062	1005	6197
4	480	26.5	4	26	30	210	120	214
						6689	1325	6284

$$S.F. = \frac{6689 + 1325}{6284} = 1.28$$

add tension 360 lb/ft

S.F. = 1.33

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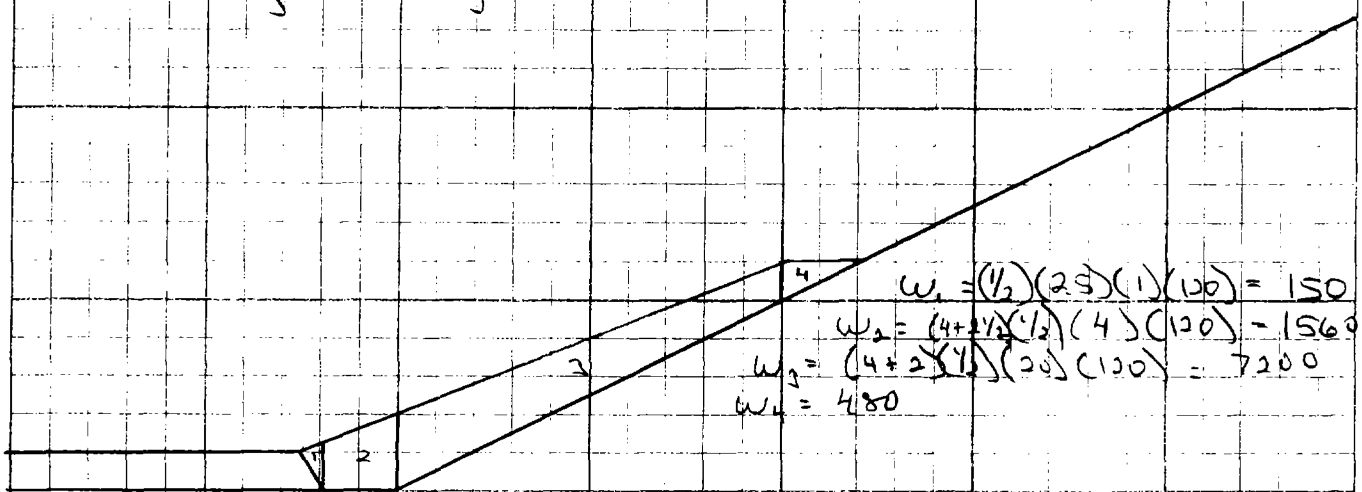
SUBJECT PC Stability SHEET 5 OF 6

Try 10' high



$W_1 = (1/2)(2.5)(1)(120) = 150$

$W_2 = (4+2\tfrac{1}{2})(1/2)(4)(120) = 1560$

$W_3 = (4+2)(1/2)(20)(120) = 7200$

$W_4 = 480$

Slice	W	α	l	ϕ	c	$W\cos\alpha\tan\phi$	cl	$W\sin\alpha$
1	150	-57.5	2	25	100	38	200	-127
2	1560	0	4	8	0	219	0	0
3	7200	26.5	22	26	30	3143	660	3213
4	480	26.5	4.5	26	30	210	120	214
						3610	980	3300

$$S.F. = \frac{3610 + 980}{3300} = 1.39$$

add tension

$$S.F. = \frac{4590 + 360}{3300} = \underline{1.5} \quad ok.$$

Check Seismic

$$S.F. = \frac{4590 + 360}{3300 + (0.0925)(9390)} = 1.19$$

would be 1.25 w/ 620 lb/ft tension

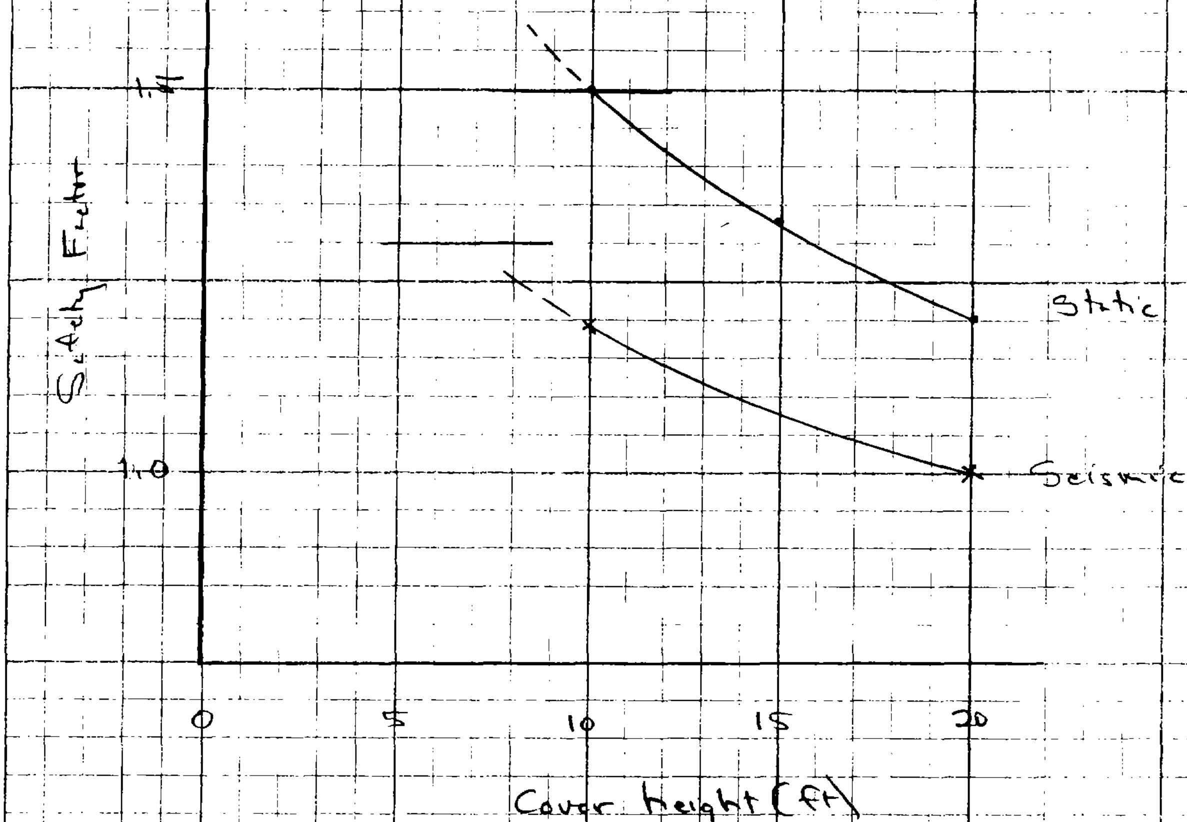
86% of yield - ok

AGEC

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/17/04 BY [illegible]

SUBJECT Cover Stability SHEET 6 OF 6



With assumed strengths

Static => 10' high

Seismic => 7' high

if we use more tension (8420) for seismic. 10' is ok.

APPENDIX 6

Settlement

AGEC

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 12/17/04 BY [illegible]

SUBJECT Settlement SHEET 1 OF 3

Settlement

Toe embankment – (17)(105) = 1785 psf

0 – 18 CL-ML
18 – 40 CL-ML (water)
40 – GM

(19.37)(2/3) + (6.4)(0.7) = 13.2" .77

Inflection Point (115)(120 psf) = 13,800 psf

0 – 18 CL-ML (6.9)(0.35) = 2.4 .23
18 – 25 CL-ML (water)
25 – GM

(27.1)(0.66) + (23)(0.7) = 26"

150' Waste = 18,000 psf

0 – 25 GM (9)(0.35) = 3.2"
25 – GM (wet)

(4.567)(0.7) = 3.2" 0.02

200' Waste = 24,000 psf

0 – 40' GM (12)(0.35) = 4.2
40 – GM (wet)

(5.675)(0.7) = 4" .02

240' Waste = 28,800 psf

0 – 50' GM 0.0007 (14.4)(0.35) = 5"
50 – GM (wet)

(7.364)(.7) = 5.15"

1"/50' of waste
12"/50' of waste

1040644

2/3

JOB NUMBER:

Constant Maximum Past Pressure: 0 psf
Length(X): 4000.0 ft Width(Y):****** ft Load:28800 psf X-Coord = .0 ft
Water Depth: 22 ft Load Depth: 0 ft Fill: 0 ft Y-Coord = .0 ft

SOIL LAYER	SOIL TYPE	LAYER THICK (FT)	LAYER DEPTH (FT)	SOIL DENSITY (PSF)	COMP RATIO	RECOMP RATIO	SETTLEMENT VIRGIN (IN)	SETTLEMENT RECOMP (IN)
1	gm	****	****	130.0	.0010	.0010	7.364	.000

TOTAL SETTLEMENT= 7.364 inches

JOB NUMBER:

Constant Maximum Past Pressure: 0 psf
Length(X): 4000.0 ft Width(Y):****** ft Load:28800 psf X-Coord = .0 ft
Water Depth: 22 ft Load Depth: 0 ft Fill: 0 ft Y-Coord = .0 ft

SOIL LAYER	SOIL TYPE	LAYER THICK (FT)	LAYER DEPTH (FT)	SOIL DENSITY (PSF)	COMP RATIO	RECOMP RATIO	SETTLEMENT VIRGIN (IN)	SETTLEMENT RECOMP (IN)
1	gm	****	****	130.0	.0010	.0010	7.364	.000

TOTAL SETTLEMENT= 7.364 inches

JOB NUMBER:

Constant Maximum Past Pressure: 0 psf
Length(X): 4000.0 ft Width(Y):4000.0 ft Load:24000 psf X-Coord = .0 ft
Water Depth: 22 ft Load Depth: 0 ft Fill: 0 ft Y-Coord = .0 ft

SOIL LAYER	SOIL TYPE	LAYER THICK (FT)	LAYER DEPTH (FT)	SOIL DENSITY (PSF)	COMP RATIO	RECOMP RATIO	SETTLEMENT VIRGIN (IN)	SETTLEMENT RECOMP (IN)
1	gm	****	****	130.0	.0010	.0010	5.675	.000

TOTAL SETTLEMENT= 5.675 inches

JOB NUMBER:

Constant Maximum Past Pressure: 0 psf
Length(X): 4000.0 ft Width(Y):4000.0 ft Load:18000 psf X-Coord = .0 ft
Water Depth: 22 ft Load Depth: 0 ft Fill: 0 ft Y-Coord = .0 ft

1040644

SOIL LAYER	SOIL TYPE	LAYER THICK (FT)	LAYER DEPTH (FT)	SOIL DENSITY (PSF)	COMP RATIO	RECOMP RATIO	SETTLEMENT VIRGIN (IN)	SETTLEMENT RECOMP (IN)
1	gm	****	****	130.0	.0010	.0010	4.567	.000

TOTAL SETTLEMENT= 4.567 inches

JOB NUMBER:

Constant Maximum Past Pressure: 0 psf
Length(X): 4000.0 ft Width(Y):4000.0 ft Load:13800 psf X-Coord = .0 ft
Water Depth: 22 ft Load Depth: 0 ft Fill: 0 ft Y-Coord = .0 ft

SOIL LAYER	SOIL TYPE	LAYER THICK (FT)	LAYER DEPTH (FT)	SOIL DENSITY (PSF)	COMP RATIO	RECOMP RATIO	SETTLEMENT VIRGIN (IN)	SETTLEMENT RECOMP (IN)
1	gm	****	****	130.0	.0010	.0010	3.721	.000

TOTAL SETTLEMENT= 3.721 inches

JOB NUMBER:

Constant Maximum Past Pressure: 0 psf
Length(X): 4000.0 ft Width(Y):4000.0 ft Load:13800 psf X-Coord = .0 ft
Water Depth: 22 ft Load Depth: 0 ft Fill: 0 ft Y-Coord = .0 ft

SOIL LAYER	SOIL TYPE	LAYER THICK (FT)	LAYER DEPTH (FT)	SOIL DENSITY (PSF)	COMP RATIO	RECOMP RATIO	SETTLEMENT VIRGIN (IN)	SETTLEMENT RECOMP (IN)
1	CL/ML	25	25	105.0	.1040	.1400	37.117	.000
2	gm	974	999	130.0	.0010	.0010	2.315	.000

TOTAL SETTLEMENT= 39.432 inches

JOB NUMBER:

Constant Maximum Past Pressure: 0 psf
Length(X): 4000.0 ft Width(Y):4000.0 ft Load: 1800 psf X-Coord = .0 ft
Water Depth: 22 ft Load Depth: 0 ft Fill: 0 ft Y-Coord = .0 ft

SOIL LAYER	SOIL TYPE	LAYER THICK (FT)	LAYER DEPTH (FT)	SOIL DENSITY (PSF)	COMP RATIO	RECOMP RATIO	SETTLEMENT VIRGIN (IN)	SETTLEMENT RECOMP (IN)
1	CL/ML	40	40	105.0	.1040	.1400	19.370	.000
2	gm	959	999	130.0	.0010	.0010	.393	.000

TOTAL SETTLEMENT= 19.763 inches

APPENDIX 7

Liquefaction

LIQUEFACTION POTENTIAL AND LIQUEFACTION INDUCED SETTLEMENT

Project No.	1040629				Salt Lake Liq. Hazard Rating		
Project Name	Wasatch Regional Landfill	Earthquake Magnitude	7.2		Liq. Potential	PGA	
Date	17-Dec-04	Magitude Scaling Factor	1.11		Very Low	>0.33g	
Time	2:16 PM	Hammer Energy Ratio	1.5		Low	0.23g-0.33g	
Site PGA for 10% in 50 yrs	0.11 g	Soil Total Unit Wt, pcf	120		Moderate	0.13g-0.23g	
Site PGA for 2% in 50 yrs	0.22 g	Hole Diameter, in	3.75	95.25 1	High	<0.13g	

Site PGA based on Frankel, A.C., Mueller, T., Barnhard, D., Perkins, E.V., Leyendecker, N., Dickman, S., Hanson, S. and Hopper, M., 1996 National Seismic Hazard Maps, U.S. Geological Survey Open-File Report 96-532

Liquefaction potential based on Youd, T. L. and I. M. Idriss, 1997, Proceedings of the NCEER workshop on liquefaction resistance of soils, Technical Report NCEER-97-0022

Liquefaction induced settlement based on Tokimatsu, A. M. and H. B. Seed, 1987, Evaluation of settlements in sands due to earthquake shaking, Journal of Geotechnical Engineering, Vol. 113, No. 8, pp. 861-878

									Corrections to SPT						LIQUEFACTION POTENTIAL							LIQUEFACTION INDUCED SETTLEMENT					
Boring	Sample Depth, ft	N	Sample Type, 1=Cal 2=SPT	% Fines	Soil Type	Water Depth, ft	Total Stress, tsf	Effective Stress, tsf	Over-burden Press., C_N	Energy Ratio, C_E	Bore Dia., C_B	Rod Length, C_R	Sample Type, C_S	Total SPT Corr.	$(N_1)_{60}$	$(N_1)_{60CS}$	$CRR_{7.5}$	CRR_M	r_d	Acc. To Cause Liq., a_L	Liq. Potential	$(N_1)_{60CS}$	10% in 50 Yr CSR	Layer Thick., ft	Vol. Strain, %	Sett., in	Cum. Sett., in
B-1	2	59	1	0	GM	136	0.120	0.120	2.00	1.5	1	0.75	0.82	1.85	108.9	108.9	0.457	0.507	1.00	0.782	Non	108.9	0.071			0.00	0.00
B-1	4	37	1	0	GM	136	0.240	0.240	2.00	1.5	1	0.75	0.82	1.85	68.3	68.3	0.457	0.507	0.99	0.786	Non	68.3	0.071			0.00	0.00
B-1	9	23	1	0	GM	136	0.540	0.540	1.36	1.5	1	0.75	0.82	1.26	28.9	28.9	0.370	0.411	0.98	0.644	Non	28.9	0.070			0.00	0.00
B-1	14	100	1	0	GM	136	0.840	0.840	1.09	1.5	1	0.80	0.82	1.08	107.5	107.5	0.457	0.507	0.97	0.804	Non	107.5	0.069			0.00	0.00
B-1	19	36	1	0	GM	136	1.140	1.140	0.94	1.5	1	0.88	0.82	1.01	36.3	36.3	0.457	0.507	0.96	0.813	Non	36.3	0.069			0.00	0.00
B-1	24	100	1	0	GM	136	1.440	1.440	0.83	1.5	1	0.93	0.82	0.95	95.0	95.0	0.457	0.507	0.95	0.825	Non	95.0	0.068			0.00	0.00
B-1	29	69	1	0	GM	136	1.740	1.740	0.76	1.5	1	0.95	0.82	0.89	61.3	61.3	0.457	0.507	0.93	0.843	Non	61.3	0.066			0.00	0.00
B-1	34	100	1	0	GM	136	2.040	2.040	0.70	1.5	1	0.96	0.82	0.83	82.7	82.7	0.457	0.507	0.90	0.869	Non	82.7	0.064			0.00	0.00
B-1	39	100	1	0	SM	136	2.340	2.340	0.65	1.5	1	0.97	0.82	0.78	77.7	77.7	0.457	0.507	0.86	0.908	Non	77.7	0.061			0.00	0.00
B-1	44	23	1	0	CL	136	2.640	2.640	0.62	1.5	1	0.97	0.82	0.74	16.9	16.9	0.183	0.203	0.81	0.384	Non	16.9	0.058			0.00	0.00
B-1	49	45	1	0	GC	136	2.940	2.940	0.58	1.5	1	0.98	0.82	0.70	31.6	31.6	0.457	0.507	0.76	1.023	Non	31.6	0.056			0.00	0.00
B-1	58	100	1	0	GC	136	3.480	3.480	0.54	1.5	1	0.99	0.82	0.66	66.3	66.3	0.457	0.507	0.66	1.154	Non	66.3	0.048			0.00	0.00
B-1	68	22	1	0	CL	136	4.080	4.080	0.50	1.5	1	1.00	0.82	0.61	13.4	13.4	0.145	0.161	0.60	0.410	Non	13.4	0.043			0.00	0.00
B-1	78	49	1	0	GC	136	4.680	4.680	0.46	1.5	1	1.00	0.82	0.57	27.9	27.9	0.339	0.377	0.56	1.043	Non	27.9	0.040			0.00	0.00
B-1	89	45	1	0	GM	136	5.340	5.340	0.43	1.5	1	1.00	0.82	0.53	24.0	24.0	0.267	0.296	0.52	0.872	Non	24.0	0.037			0.00	0.00
B-1	98	100	1	0	GC	136	5.880	5.880	0.41	1.5	1	1.00	0.82	0.51	50.7	50.7	0.457	0.507	0.50	1.553	Non	50.7	0.036			0.00	0.00
B-1	108	77	1	0	GC	136	6.480	6.480	0.39	1.5	1	1.00	0.82	0.48	37.2	37.2	0.457	0.507	0.49	1.607	Non	37.2	0.035			0.00	0.00
B-1	118	45	1	0	GC	136	7.080	7.080	0.38	1.5	1	1.00	0.82	0.46	20.8	20.8	0.226	0.250	0.47	0.817	Non	20.8	0.034			0.00	0.00
B-1	128	100	1	0	GC	136	7.680	7.680	0.36	1.5	1	1.00	0.82	0.44	44.4	44.4	0.457	0.507	0.46	1.699	Non	44.4	0.033			0.00	0.00
B-1	138	100	1	0	GC	136	8.280	8.218	0.35	1.5	1	1.00	0.82	0.43	42.9	42.9	0.457	0.507	0.45	1.727	Very Low	42.9	0.032			0.00	0.00
B-2	0.5	25	1	0	ML	102	0.030	0.030	2.00	1.5	1	0.75	0.82	1.85	46.1	46.1	0.457	0.507	1.00	0.780	Non	46.1	0.072			0.00	0.00
B-2	4	68	1	0	GM	102	0.240	0.240	2.00	1.5	1	0.75	0.82	1.85	125.5	125.5	0.457	0.507	0.99	0.786	Non	125.5	0.071			0.00	0.00
B-2	9	54	1	0	GC	102	0.540	0.540	1.36	1.5	1	0.75	0.82	1.26	67.8	67.8	0.457	0.507	0.98	0.795	Non	67.8	0.070			0.00	0.00
B-2	14	52	1	0	SM	102	0.840	0.840	1.09	1.5	1	0.80	0.82	1.08	55.9	55.9	0.457	0.507	0.97	0.804	Non	55.9	0.069			0.00	0.00
B-2	19	100	1	0	GM	102	1.140	1.140	0.94	1.5	1	0.88	0.82	1.01	101.0	101.0	0.457	0.507	0.96	0.813	Non	101.0	0.069			0.00	0.00
B-2	24	77	1	0	GM	102	1.440	1.440	0.83	1.5	1	0.93	0.82	0.95	73.2	73.2	0.457	0.507	0.95	0.825	Non	73.2	0.068			0.00	0.00
B-2	29	56	1	0	GM	102	1.740	1.740	0.76	1.5	1	0.95	0.82	0.89	49.8	49.8	0.457	0.507	0.93	0.843	Non	49.8	0.066			0.00	0.00
B-2	34	7	1	64	SM	102	2.040	2.040	0.70	1.5	1	0.96	0.82	0.83	5.8	11.9	0.129	0.143	0.90	0.246	Non	11.9	0.064			0.00	0.00
B-2	39	37	1	0	GC	102	2.340	2.340	0.65	1.5	1	0.97	0.82	0.78	28.7	28.7	0.365	0.406	0.86	0.726	Non	28.7	0.061			0.00	0.00
B-2	44	100	1	0	GM	102	2.640	2.640	0.62	1.5	1	0.97	0.82	0.74	73.6	73.6	0.457	0.507	0.81	0.959	Non	73.6	0.058			0.00	0.00
B-2	49	100	1	0	GM	102	2.940	2.940	0.58	1.5	1	0.98	0.82	0.70	70.2	70.2	0.457	0.507	0.76	1.023	Non	70.2	0.055			0.00	0.00
B-2	54	18	1	0	SC	102	3.240	3.240	0.56	1.5	1	0.99	0.82	0.67	12.1	12.1	0.131	0.146	0.71	0.314	Non	12.1	0.051			0.00	0.00
B-2	59	100	1	0	SM	102	3.540	3.540	0.53	1.5	1	0.99	0.82	0.65	64.8	64.8	0.457	0.507	0.67	1.169	Non	64.8	0.048			0.00	0.00
B-2	64	100	1	0	GM	102	3.840	3.840	0.51	1.5	1	1.00	0.82	0.63	62.6	62.6	0.457	0.507	0.63	1.241	Non	62.6	0.046			0.00	0.00
B-2	69	100	1	0	GM	102	4.140	4.140	0.49	1.5	1	1.00	0.82	0.60	60.5	60.5	0.457	0.507	0.60	1.306	Non	60.5	0.043			0.00	0.00
B-2	74	100	1	0	GM	102	4.440	4.440	0.47	1.5	1	1.00	0.82	0.58	58.4	58.4	0.457	0.507	0.57	1.364	Non	58.4	0.041			0.00	0.00
B-2	79	100	1	0	GC	102	4.740	4.740	0.46	1.5	1	1.00	0.82	0.56	56.5	56.5	0.457	0.507	0.55	1.414	Non	56.5	0.039			0.00	0.00
B-3	0.5	15	1	0	CL	13	0.030	0.030	2.00	1.5	1	0.75	0.82	1.85	27.7	27.7	0.336	0.372	1.00	0.671	Non	27.7	0.072			0.00	0.00
B-3	4	18	1	0	ML	13	0.240	0.240	2.00	1.5	1	0.75	0.82	1.85	33.2	33.2	0.457	0.507	0.99	0.786	Non	33.2	0.071			0.00	0.00
B-3	9	9	1	0	CL	13	0.540	0.540	1.36	1.5	1	0.75	0.82	1.26	11.3	11.3	0.122	0.136	0.98	0.213	Non	11.3	0.070			0.00	0.00
B-3	14	33	1	0	SM	13	0.840	0.809	1.11	1.5	1	0.80	0.82	1.10	36.2	36.2	0.457	0.507	0.97	0.774	Very Low	36.2	0.072			0.00	0.00
B-3	19	42	1	0	SM	13	1.140	0.953	1.02	1.5	1	0.88	0.82	1.10	46.3	46.3	0.457	0.507	0.96	0.680	Very Low	46.3	0.082			0.00	0.00
B-3	24	37	1	0	SM	13	1.440	1.097	0.95	1.5	1	0.93	0.82	1.09	40.3	40.3	0.457	0.507	0.95	0.629	Very Low	40.3	0.089			0.00	0.00
B-3	29	6	1	0	CL	13	1.740	1.241	0.90	1.5	1	0.95	0.82	1.06	6.3	6.3	0.074	0.083	0.93	0.098	Non	6.3	0.093			0.00	0.00
B-3	34	3	1	0	CL	13	2.040	1.385	0.85	1.5	1	0.96	0.82	1.00	3.0	3.0	0.055	0.061	0.90	0.072	Non	3.0	0.095			0.00	0.00
B-4	2	6	1	0	SM	63	0.120	0.120	2.00	1.5	1	0.75	0.82	1.85	11.1	11.1	0.120	0.133	1.00	0.206	Non	11.1	0.071			0.00	0.00
B-4	4	35	1	0	GC	63	0.240	0.240	2.00	1.5	1	0.75	0.82	1.85	64.6	64.6	0.457	0.507	0.99	0.786	Non	64.6	0.071			0.00	0.00
B-4	9	12	1	0	SM	63	0.540	0.540	1.36	1.5	1	0.75	0.82	1.26	15.1	15.1	0.163	0.181	0.98	0.283	Non	15.1	0.070			0.00	0.00
B-4	14	4	1	50	SM	63	0.840	0.840	1.09	1.5	1	0.80	0.82	1.08	4.3	10.2	0.110	0.122	0.97	0.194	Non	10.2	0.069			0.00	0.00
B-4	19	5	1	66	ML	63	1.140	1.140	0.94	1.5	1	0.88	0.82	1.01	5.0	11.1	0.120	0.133	0.96	0.213	Non	11.1	0.069			0.00	0.00
B-4	24	7	1	16	SM	63	1.440	1.440	0.83	1.5	1	0.93	0.82	0.95	6.7	9.5	0.103	0.114	0.95	0.186	Non	9.5	0.068			0.00	0.00
B-4	29	100	1	0	SP	63	1.740	1.740	0.76	1.5	1	0.95	0.82	0.89	88.9	88.9	0.457	0.507	0.93	0.843	Non	88.9	0.066			0.00	0.00
B-4	34	100	1	0	GC	63	2.040	2.040	0.70	1.5	1	0.96	0.82	0.83	82.7	82.7	0.457	0.507	0.90	0.869	Non	82.7	0.064			0.00	0.00
B-4	39	40	1	0	GC	63	2.340	2.340	0.65	1.5	1	0.97	0.82	0.78	31.1	31.1	0.457	0.507	0.86	0.906	Non	31.1	0.061			0.00	0.00
B-4	44	100	1	0	GC	63	2.640	2.640	0.62	1.5	1	0.97	0.82	0.74	73.6	73.6	0.457	0.507	0.81	0.959	Non	73.6	0.058			0.00	0.00
B-4	49	78	1	0	GC	63	2.940	2.940	0.58	1.5	1	0.98	0.82	0.70	54.8	54.8	0.457	0.507	0.76	1.023	Non	54.8	0.055			0.00	0.00
B-4	54	100	1	0	GC	63	3.240	3.209	0.56	1.5	1	0.99	0.82	0.68	67.7	67.7	0.457	0.507	0.71	1.084	Very Low	67.7	0.051			0.00	0.00
B-4	59	28	1	0	SC	63	3.540	3.353	0.55	1.5	1	0.99	0.82	0.67	18.7	18.7	0.201	0.223	0.67	0.487	Very Low	18.7	0.050			0.00	0.00
B-4	64	100	1	0	GC	63	3.840	3.497	0.53	1.5	1	1.00	0.82	0.66	65.6	65.6	0.457	0.507	0.63	1.130	Very Low	65.6	0.049			0.00	0.00
B-4	69	58	1	0	GC	63	4.140	3.641	0.52	1.5	1	1.00	0.82	0.64	37.4	37.4	0.457	0.507	0.60	1.149	Very Low	37.4	0.049			0.00	0.00
B-4	73	61	1	0	GC	63	4.380	3.756	0.52	1.5	1	1.00	0.82	0.63	38.7	38.7	0.457	0.507	0.58	1.160	Very Low	38.7	0.048			0.00	0.00
B-4	78	32	1	0	GC	63	4.680	3.900	0.51	1.5	1	1.00	0.82	0.62	19.9	19.9	0.216	0.239	0.56	0.562	Very Low	19.9	0.048			0.00	0.00
B-5	2	5	1	0	CL	13	0.120	0.120	2.00	1.5	1	0.75	0.82	1.85	9.2	9.2	0.101	0.112	1.00	0.172	Non	9.2	0.071			0.00	0.00
B-5	4	6	1	0	CL	13	0.240	0.240	2.00	1.5	1	0.75	0.82	1.85	11.1	11.1	0.120	0.133	0.99	0.206	Non	11.1	0.071			0.00	0.00
B-5	9	7	1	0	CL	13	0.540	0.540	1.36	1.5	1	0.75	0.82	1.26	8.8	8.8	0.096	0.107	0.98	0.167	Non	8.8	0.070			0.00	0.00
B-5	14	25	1	0	SM	13	0.840	0.809	1.11	1.5	1	0.80	0.82	1.10	27.4	27.4	0.328	0.364	0.97	0.556	Very Low	27.4	0.072			0.00	0.00
B-5	19	45	1	0	SM	13	1.140	0.953	1.02	1.5	1	0.88	0.82	1.10	49.7	49.7	0.457	0.507	0.96	0.680	Very Low	49.7	0.082			0.00	0.00
B-5	24	40	1	0	SM	13	1.440	1.097	0.95	1.5	1	0.93	0.82	1.09	43.6	43.6	0.457	0.507	0.95	0.629	Very Low	43.6	0.089			0.00	0.00
B-5	29	12	1	28	SM	13	1.740	1.241	0.90	1.5	1	0.95	0.82	1.05	12.6	18.9	0.204	0.227	0.93	0.289	Very Low	18.9	0.093			0.00	0.00
B-5	34	3	1	88	CL	13	2.040	1.385	0.85	1.5	1	0.96	0.82	1.00	3.0	8.6	0.094	0.105	0.90	0.122	Non	8.6	0.095			0.00	0.00

Applied Geotechnical Engineering Consultants, P.C.

May 10, 2005

Wasatch Regional Landfill
c/o Hansen, Allen and Luce, Incorporated
6771 South 900 East
Midvale, UT 84047

Attention: Kent Staheli
FAX: 566-5581

Subject: Response to Request for Additional Information, No. 1 (April 22, 2005)
Wasatch Regional Solid Waste Class V Landfill
Permit Modification Review
Tooele County, Utah
AGEC Project No. 1040644

Applied Geotechnical Engineering Consultants, P.C. (AGEC) was requested to provide additional information requested by the Utah Solid and Hazardous Waste Control Board for the modification to the Wasatch Regional Solid Waste Class V Landfill Permit modification.

AGEC previously conducted a geotechnical investigation for the proposed modification and presented our findings and recommendations in a report dated December 17, 2004 under Project No. 1040644.

INFORMATION REQUESTED

The letter dated April 22, 2005 (from the Utah Solid and Hazardous Waste Control Board) requests additional information on two issues that pertain to the geotechnical aspects of the modification. The additional information is requested in their Comments Nos. 14 and 15.

Item No. 14

> Page 14 states, "This acceleration was adjusted for the stability analysis as recommended in the DMG Special Publication 117 (Guidelines for Analyzing and Mitigating Landslide Hazards in California). Using this document, an acceleration of 0.092g was used for the stability calculations assuming a threshold of 15 cm displacement".

Comment

The staff has used the RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities. However, the staff is not familiar with Publication 117. A copy of the publication needs to be included in the modification with a discussion of how it was applied in the model.

Response

As requested, a copy of DMG Special Publication 117 is attached.

Publication 117 was used to determine the factor, that may be applied to the maximum horizontal ground acceleration, in order to determine the horizontal coefficient that may be used in the pseudo-static stability analysis. The figure, from which the reduction factor was obtained, is included on the above referenced report on Page 10/14 within Appendix 4 (Landfill Stability). This same figure is located on Page 81 of Special Publication 117.

A factor of 0.44 was applied to the maximum acceleration to determine the horizontal acceleration coefficient with a 15 cm threshold of displacement.

Impact of the Seismic Coefficient

Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities references two methods to estimate the potential movement based on the ratio of the yield acceleration compared to the maximum design acceleration. As indicated on attached sheet 4 of 5, this ratio ranges from 0.44 to greater than 1 for the landfill. A value greater than one indicates that there would be no movement under the influence of the design acceleration. The lowest ratios (0.44 and 0.57) would indicate the potential for 17 cm (upper bound using Hynes & Franklin) to 33 cm (upper bound of Makdisi & Seed) of displacements.

The analyses with potential displacement are for the floor (17 cm) using an assumed weak strength between the HDPE and the GCL of 8 degrees. The other potential displacement (33 cm) is on the interior soil protective cover using only 50% of the available tension in the synthetic materials.

Including the analysis using the DMG Publication, it is our professional opinion that the potential displacements during a major seismic event (the design event) will be less than those estimated above due to the anticipated strengths that will most likely apply after construction (our analysis has assumed conservative strengths). Therefore, it is also our professional opinion that the landfill, as currently designed, will meet the intent of the design guidance for municipal waste landfill.

Item No. 15

Page 15 states, "The testing consisted of penetration resistances, unconfined compressive strength tests, triaxial shear tests and direct shear tests conducted on undisturbed and remolded soil samples. Based on these results, previous testing by others and our judgement, strength parameters for each material were selected.

Comment

Specific reference to test results and supporting data need to be provided to support each one of the selected parameters. As one example, strength parameters provided on Page 15 show the unit weight for waste is 120 pounds per cubic foot. The Class 5 permit application used a unit weight of 72.6 pounds per cubic foot for waste. The modification needs to include the justification for using another number.

Response

The values used for unit weight, friction and cohesion for each of the materials included in our analysis are presented in Appendix 1 of the geotechnical report (Soil Characteristics). Listed below is a summary of each of the parameters used and the source of the information.

<u>Waste</u>

a. Unit weight of 120 pounds per cubic foot

The 120 pounds per cubic foot weight for waste for was simply selected as a high value, which essentially models soil with no waste. The value included in the permit application (72.6 pounds per cubic foot) is higher than what is referenced (46 to 65 pounds per cubic foot - page 103 - Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities). The higher weight used in our analysis is conservative in that it provides a larger driving force downslope, a higher horizontal component during the seismic analysis (acceleration time the unit weight) but, also provides a higher resistance (less conservative) to sliding for frictional contacts. In order to demonstrate the impact of using 120 pcf, 72.6 pcf and 65 pcf, the landfill stability was evaluated with each of these parameters. The results are indicated below:

Unit Weight (pcf)	Static Safety Factor	Seismic Safety Factor (a = 0.21g)
65	2.478	1.225
72.6	2.452	1.212
120	2.363	1.163

As indicated by this analysis, the use of 120 pounds per cubic foot is conservative with the design.

Waste Strengths

A friction value of 25 degrees and a cohesion of 100 pounds per cubic foot were used for the strength characteristics of the waste materials. As indicated in the guidance document, the friction and the cohesion values used correspond with the lowest values included in Table 6.3 (lower bound friction angles back figured from observations of steep landfill slopes, as indicated on Page 117 of the RCRA Subtitle D (258) Seismic Design Guidance for Municipal Solid Waste Landfill Facilities. Using the lowest values will provide the more conservative analysis.

Embankment Materials

The embankment material unit weight is close to the average of on-site materials compacted to 95 percent of the maximum dry density at the optimum moisture

The strength parameters used are less than the values obtained from the laboratory tests on remolded samples of the fine-grained soil. The laboratory tests indicate a friction angle of 35 degrees with a cohesion intercept of 550 pounds per square foot. For our analysis, we have used a friction angle of 32 degrees and a cohesion of 300 pounds per square foot, (60 to 89 percent of the laboratory values).

Foundation Soil

An average unit weight of 105 pcf was used for the fine-grained foundation soil. This density is based on the typical values obtained from laboratory tests. The density is based on the typical values obtained from laboratory tests. The values can be seen on Sheet 4 of 6 of Appendix 1 of the geotechnical report.

The strength of the fine-grained soil was tested in the laboratory. The results are summarized on Sheet 3/6 within Appendix 1 (Soil Characteristics). An average friction angle of 31.6 degrees and an average cohesion of 43 pounds per square foot were obtained. With these values, we have used a friction angle of 31 degrees and a cohesive intercept of 40 pounds per square foot, (93 to 98 percent of the laboratory average).

Natural Gravel

A unit weight of 130 pounds per cubic foot for the gravel was used in our analysis. This value is slightly less than the value obtained in the laboratory. The values obtained are shown on Sheet 4 of 6 of Appendix 1 (Soil Characteristics) of the geotechnical report.

The strength of the granular soil was determined by evaluating the penetration resistance values (Sheet 5 of 6, Appendix 1) along with correlation of penetration resistance versus friction angle. The values obtained during our study was significantly greater than those obtained by Kleinfelder. It is our professional opinion that the higher values are due to the fact that our borings were further up the hill, sampling denser material. A friction value of 37 degrees was, therefore, selected and used in the analysis.

It is our professional opinion that the values used in the analysis are representative of the materials that will be in place and used during construction. These values are appropriate for modeling of the conditions that will be experienced.

If you have any questions or we can be of further service, please call.

Sincerely,

APPLIED GEOTECHNICAL ENGINEERING CONSULTANTS, P.C.

REGISTERED PROFESSIONAL ENGINEER
5/10/05
STATE OF UTAH

James E. Nordquist, P.E.
JEN/sc
Enclosures

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 5/7/05 BY [illegible]

SUBJECT Stability w/ Waste weights (different) SHEET 1 OF

Overall landfill stability

- As previously presented at 120 pcf waste unit weight

File	Condition	S.F.
WRL.I9	Static w/ Waste γ = 65 pcf.	2.496
WRL.I10	" " γ = 72.6 pcf	2.465
WRL.I11	" " γ = 120 pcf	2.353
WRL.I12	Dynamic, a = 0.11 g; Waste 120 pcf	1.157
WRL.I13	" " " 72.6 pcf	1.214
WRL.I14	" " " 65 pcf	1.227

Summary - The 120 pcf is more conservative.

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 5/7/05 BY [illegible]

SUBJECT Seismic SHEET 2 OF

Three conditions resulted in Seismic S.F. ~ 1.3.

Floor

$$S.F. = \frac{\cos 0.97}{\sin 0.97 + K \cos 0.97} \tan 8°$$

w/K = 0.0925 S.F. = 1.28
= 0.21 S.F. = 0.62
= 0.12 S.F. = 1.0

ratio of $K_y/K_{max} = \frac{0.12}{0.21} = 0.57$

Exterior Side w/o cohesion

$$S.F. = \frac{\cos 14.04}{\sin 14.04 + K \cos 14.04} \tan 23.9$$

w/K = 0.0925 S.F. = 1.29
= 0.21 S.F. = 0.96
= 0.19 S.F. = 1.0

ratio of $K_y/K_{max} = 0.19/0.21 = 0.92$

Interior Cover w/ 50% of geosynthetic tensile

@ 10' K_{yield} = 0.18 ratio = 0.18/0.21 = 0.84
@ 20' K_{yield} = 0.0925 ratio = = 0.44

Applied Geotechnical Engineering Consultants, P.C.

PROJECT NO. 1040644 TITLE WRL DATE 5/17/05 BY [illegible]

SUBJECT Seismic SHEET 3 OF

Summary

Location		Acceleration Yield	Max	Y/m	Makdisi	Hynes Mean+σ	Upper Bound
Entire Landfill		>0.21	0.21	>1	0	0	0
Floor		0.12	0.21	0.57	2 - 15 cm	<10cm	$\boxed{17cm}$
Exterior Side w/o cohesion		0.19	0.21	0.90	$\boxed{0.05 - 0.3\ cm}$	<10cm	<10cm
Interior Cover 50% tension	10'	0.18	0.21	0.86	0.1 - 0.65 cm	<10cm	$\boxed{<10cm}$
	20'	0.0925	0.21	0.44	4 - $\boxed{33\ cm}$	<10cm	26cm

Sheet 4/

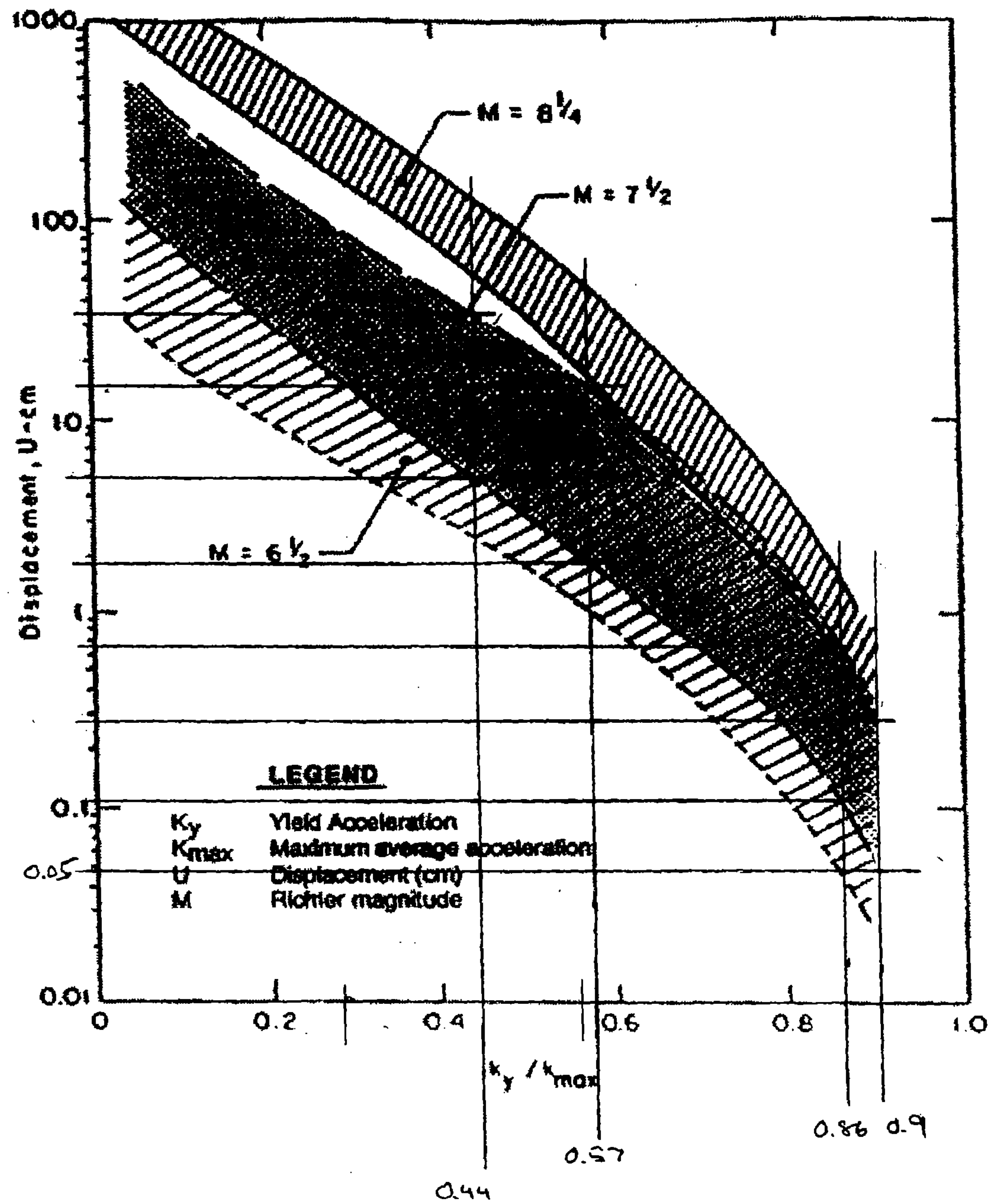


Figure 6.6 Makdisi and Seed Permanent Displacement Chart (Makdisi and Seed, 1978).

Sheet 51

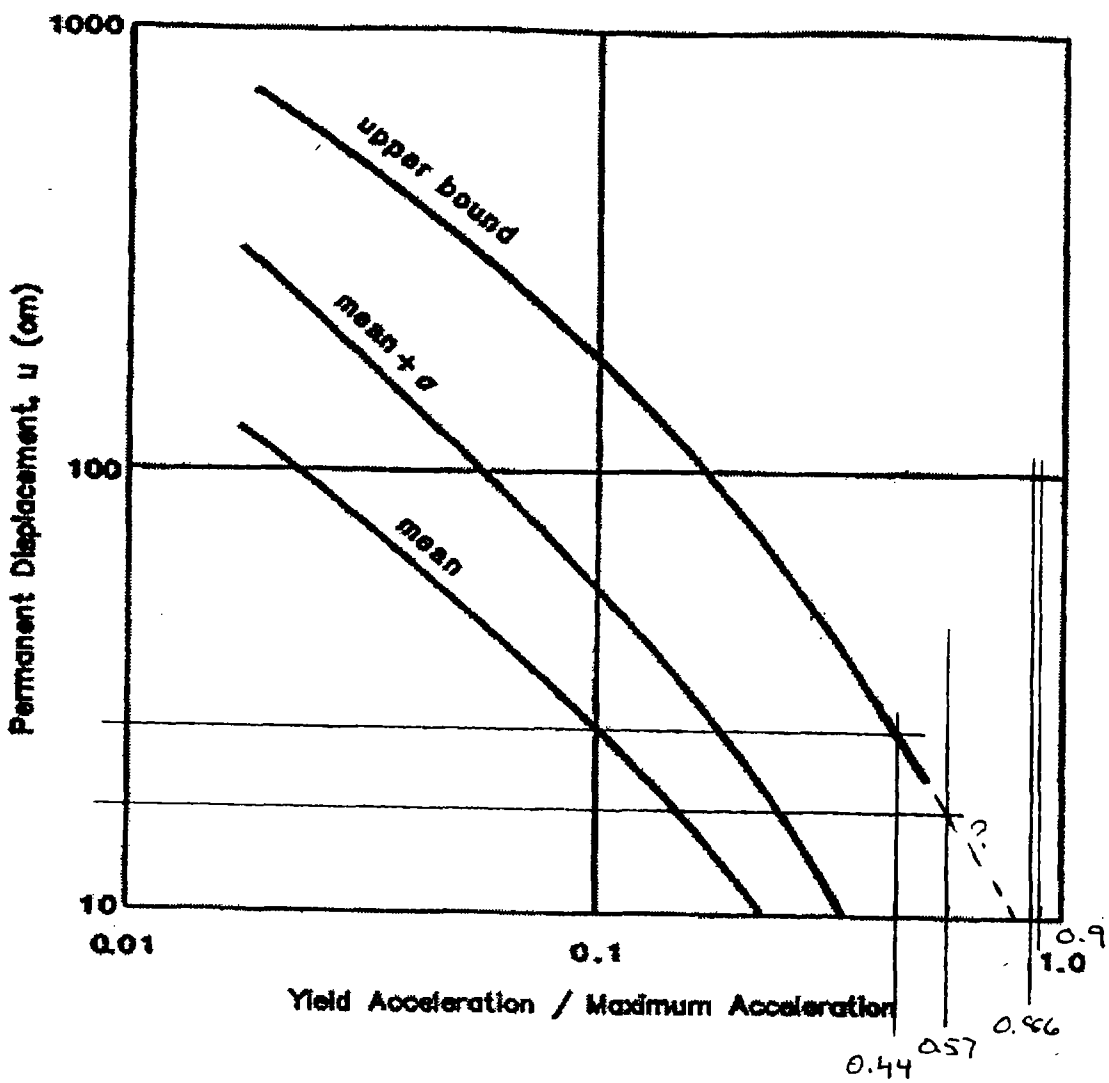


Figure 6.5 Hynes and Franklin Permanent Seismic Displacement Chart (Hynes and Franklin, 1984).

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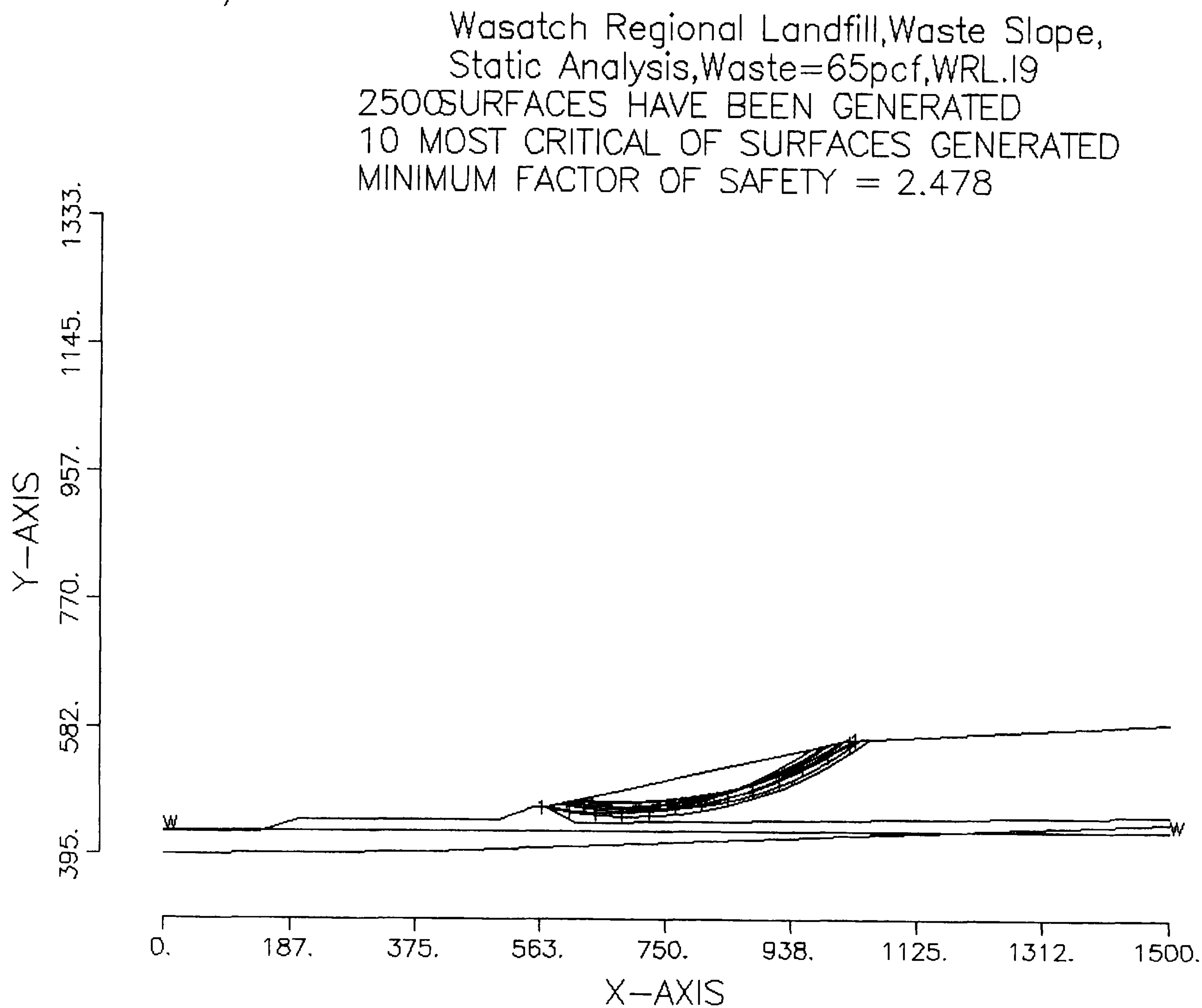

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1021. 565. 1500. 590. 1
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613. 444. 1500. 453. 2
0. 395. 400. 400. 3
400. 400. 1500. 443. 3
SOIL
3
65. 65. 100. 25. 0. 0. 1
105. 105. 40. 31. 0. 0. 1
130. 130. 0. 37. 0. 0. 1
WATER
1 62.4
2
0. 430.
1500. 430.
CIRCL2
50 50 450. 800. 950. 1400.
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END
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AGI
Midvale UT s/n5206

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atic Analysis,Waste=72.6pcf,WRL.I10
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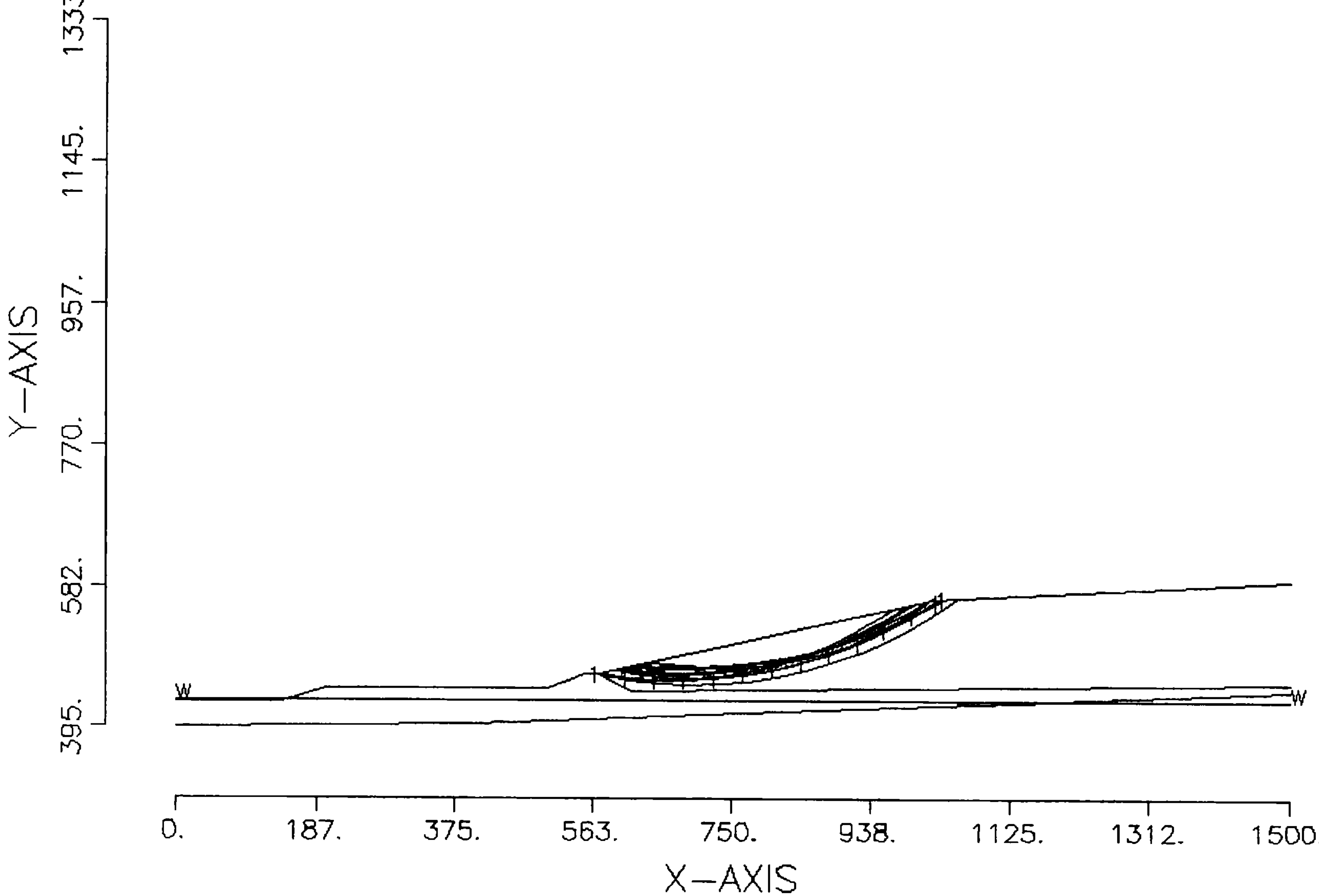
10 MOST CRITICAL OF SURFACES GENERATED
MINIMUM FACTOR OF SAFETY = 2.452



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613. 444. 1500. 453. 2
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SOIL
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72.6 72.6 100. 25. 0. 0. 1
105. 105. 40. 31. 0. 0. 1
130. 130. 0. 37. 0. 0. 1
WATER
1 62.4
2
0. 430.
1500. 430.
CIRCL2
50 50 450. 800. 950. 1400.
1. 40. 0. 0.
END
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AGF
Midvale UT s/n5206

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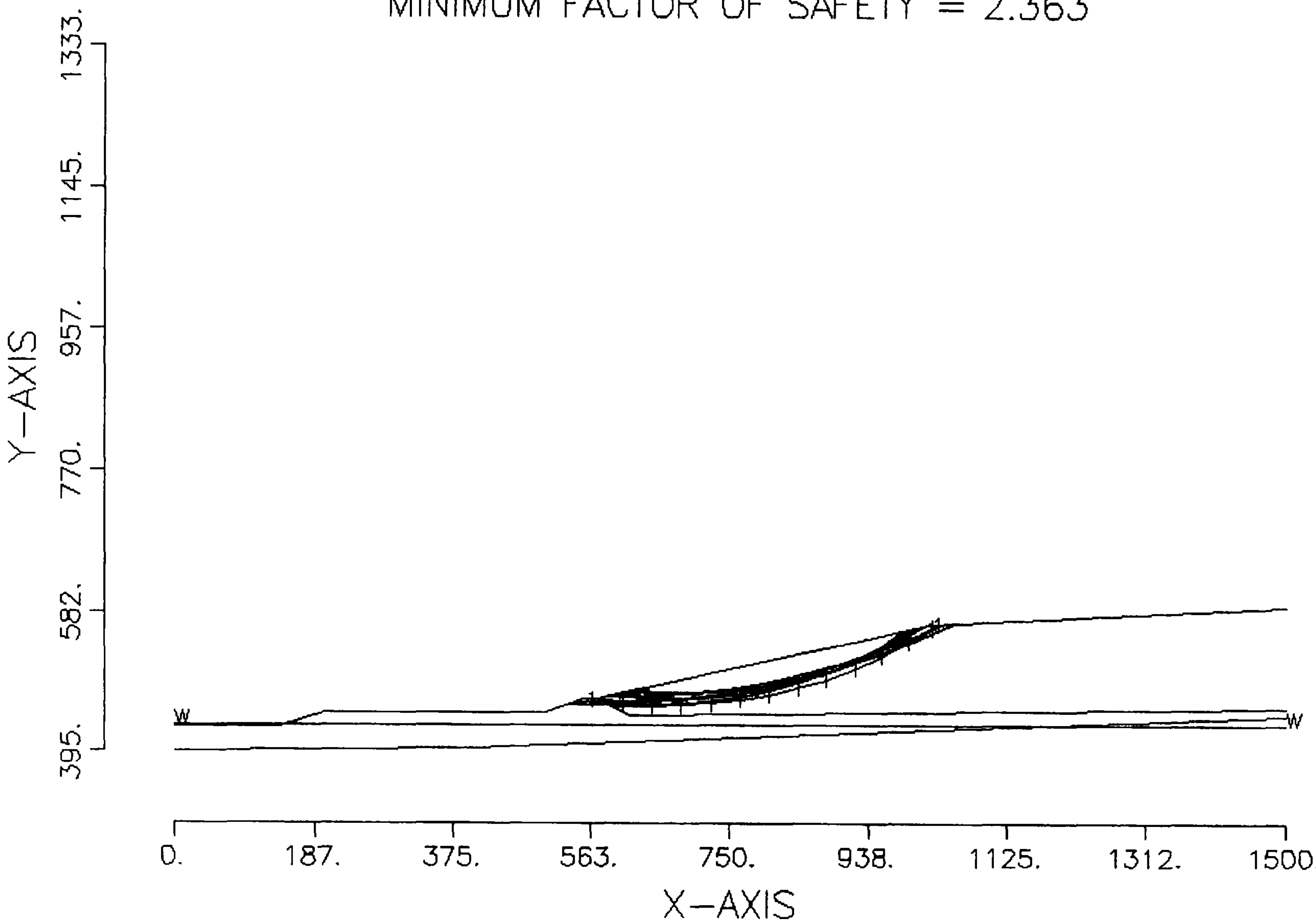

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1021. 565. 1500. 590. 1
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613. 444. 1500. 453. 2
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400. 400. 1500. 443. 3
SOIL
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105. 105. 40. 31. 0. 0. 1
130. 130. 0. 37. 0. 0. 1
WATER
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CIRCL2
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1. 40. 0. 0.
END
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AGF
Midvale UT s/n5206

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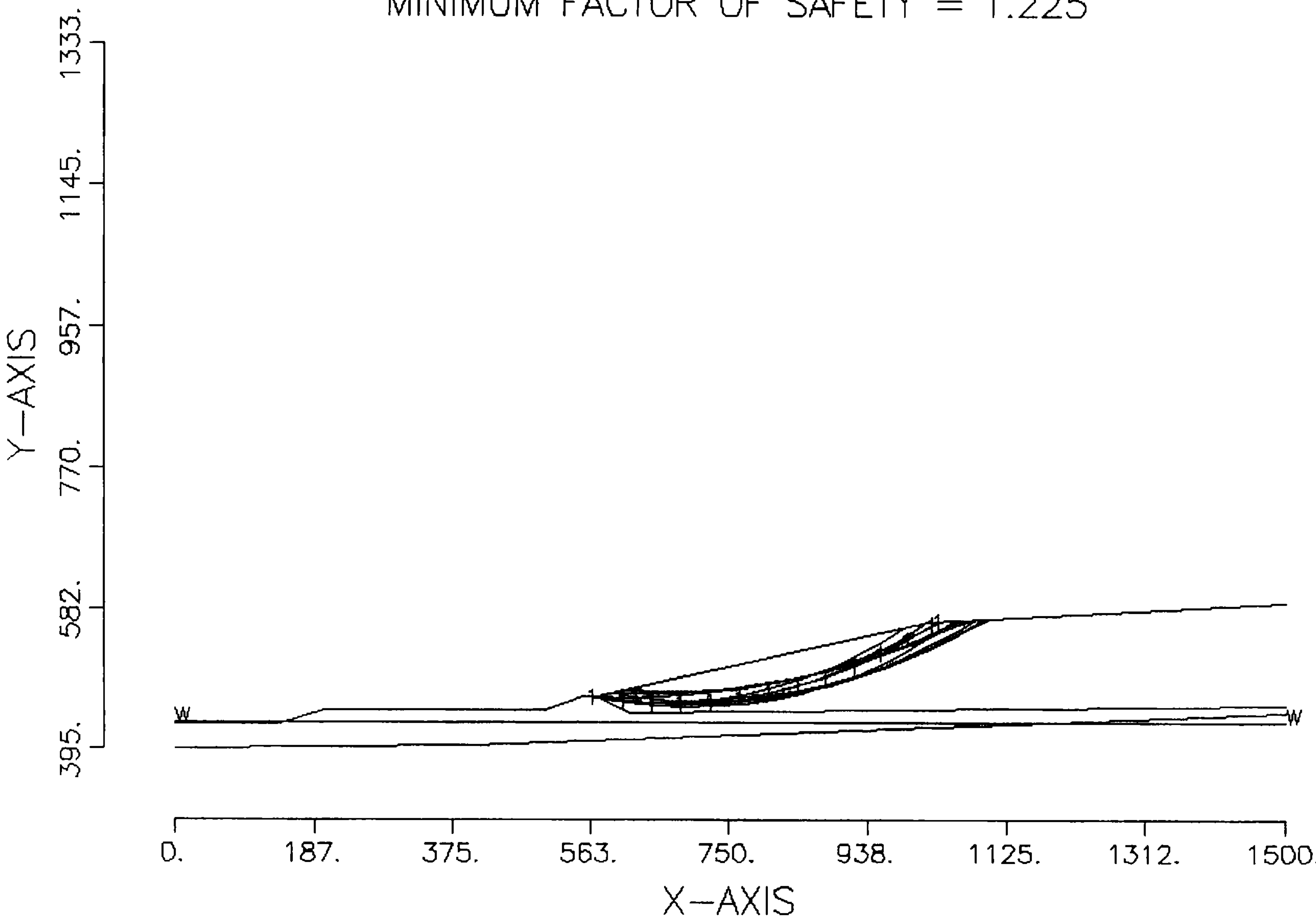

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SOIL
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WATER
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CIRCL2
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1 40. 0. 0.
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AGF
Midvale UT s/n5206

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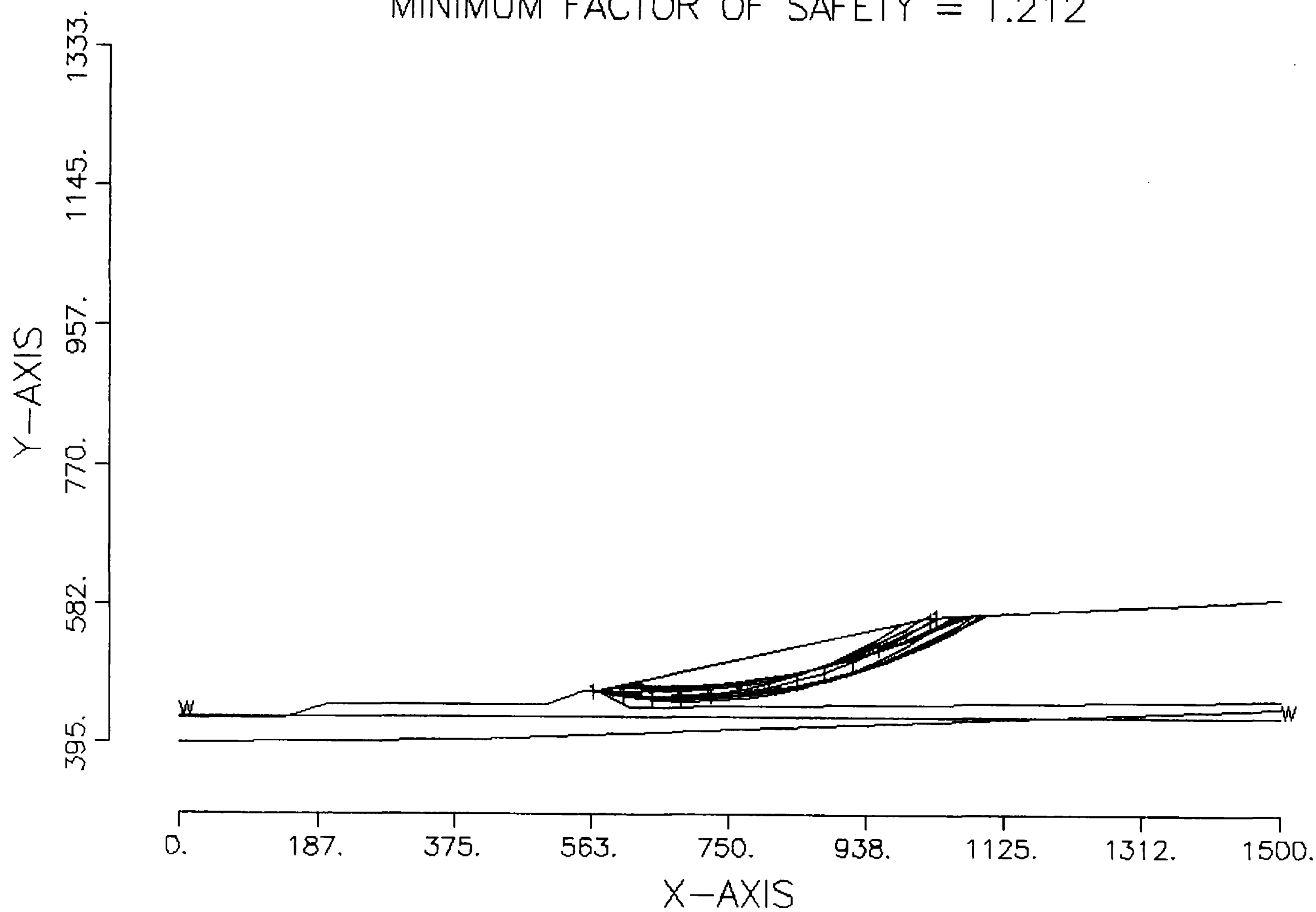

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613. 444. 1500. 453. 2
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400. 400. 1500. 443. 3
SOIL
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WATER
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EQUAKE
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AGF
Midvale UT s/n5206

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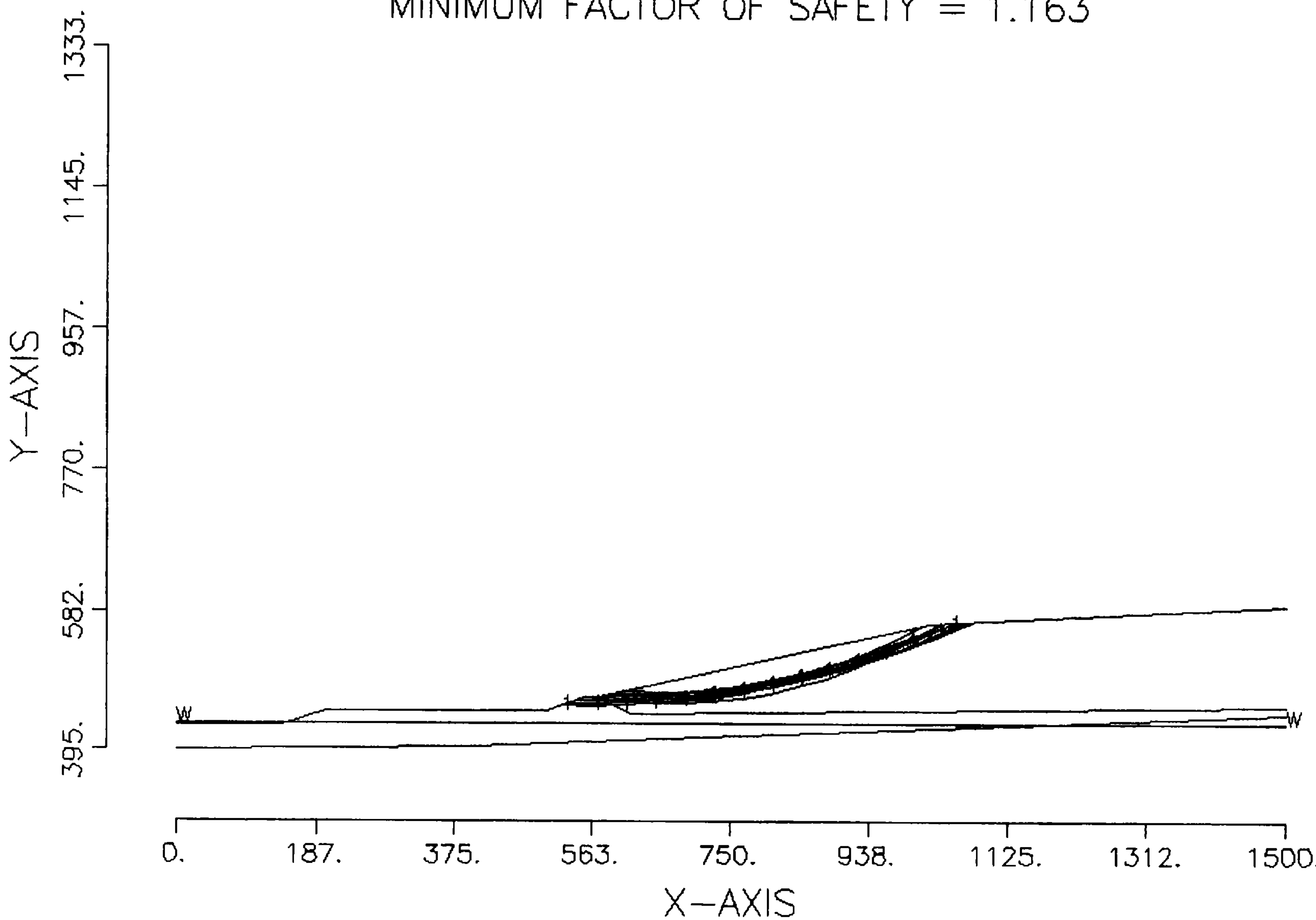

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571. 465. 613. 444. 2
613. 444. 1500. 453. 2
0. 395. 400. 400. 3
400. 400. 1500. 443. 3
SOIL
3
120. 120. 100. 25. 0. 0. 1
105. 105. 40. 31. 0. 0. 1
130. 130. 0. 37. 0. 0. 1
WATER
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EQUAKE
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1 40. 0. 0.
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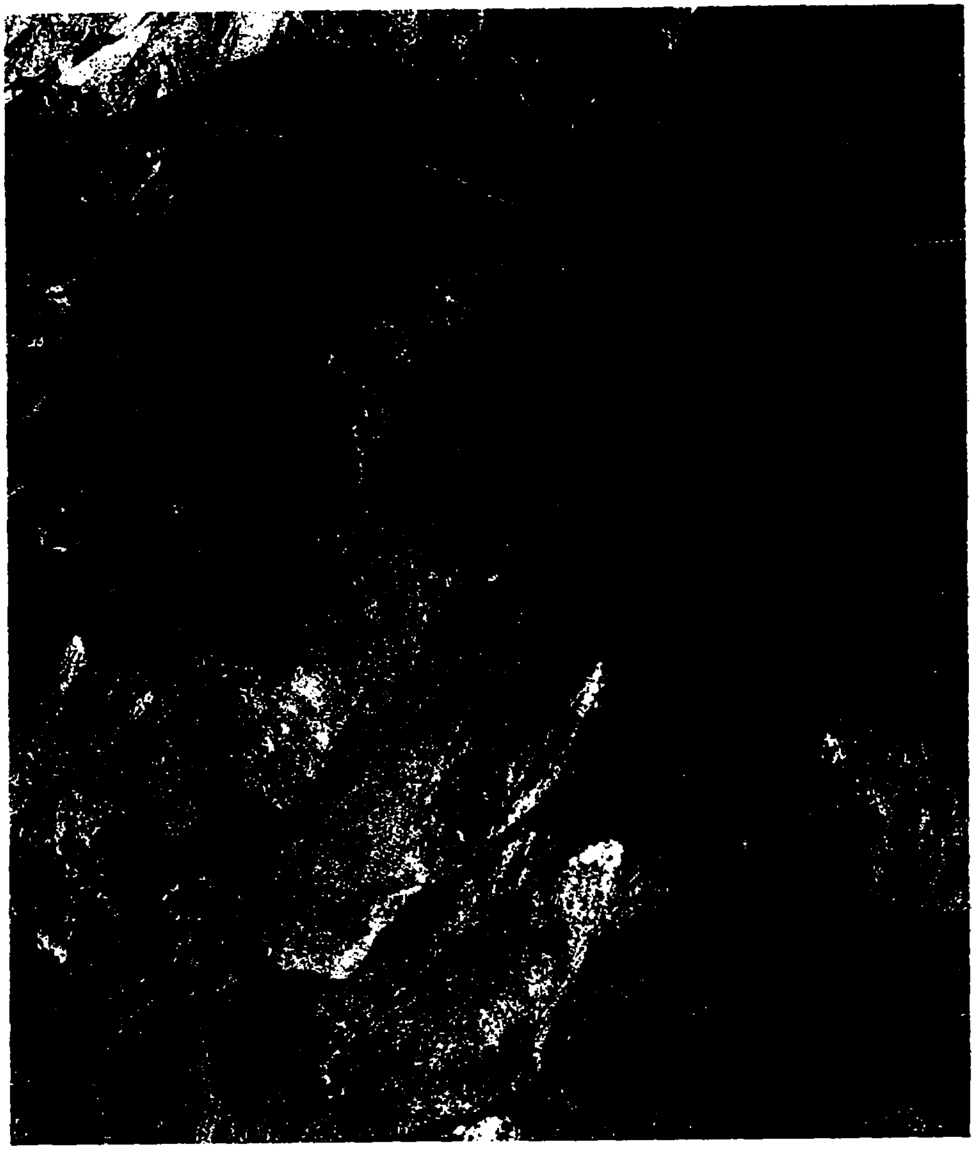

RECOMMENDED PROCEDURES FOR IMPLEMENTATION OF DMG SPECIAL PUBLICATION 117 GUIDELINES FOR ANALYZING AND MITIGATING LANDSLIDE HAZARDS IN CALIFORNIA

ASCE

Committee organized through the
ASCE Los Angeles Section Geotechnical Group
Document published by the
Southern California Earthquake Center

SCEC

Publication of this document was funded by the Southern California Earthquake Center.

The Southern California Earthquake Center (SCEC), headquartered at the University of Southern California, is a regionally focused organization founded in 1991 with a mission to gather new information about earthquakes in Southern California, integrate knowledge into a comprehensive and predictive understanding of earthquake phenomena, and communicate that understanding to end-users and the general public in order to increase earthquake awareness, reduce economic losses, and save lives. Funding for SCEC activities is provided by the National Science Foundation (NSF) and the U.S. Geological Survey (USGS). An outstanding community of scientists from over 40 institutions throughout the country participates in SCEC. The SCEC Communication, Education, and Outreach Program offers student research experiences, web-based education tools, classroom curricula, museum displays, public information brochures, online newsletters, and technical workshops and publications.

The cover photograph depicts a landslide that developed in the Ramona oilfield, north of San Martinez Grande Canyon, about 9 km east-northeast of Piru, California. The landslide is 600 m long, 100-150 m wide, and has an estimated volume of about 1 million cubic meters. During the Northridge earthquake (January 17, 1994), the landslide moved downslope about 15-25 meters. (Photograph courtesy of Randall Jibson, U.S. Geological Survey)

June 2002

The over 3-1/2 years effort of the committee members to study, evaluate, discuss, and formulate these guidelines is greatly appreciated. The summation of those consensus efforts is presented in this report.

The committee was organized by the southern California section of the Association of Civil Engineers and the City and County of Los Angeles Departments of Building and Safety and Public Works. The committee has, however, performed its work independent of those entities. The document represents the work of the committee. Although the document has been peer reviewed, the information and opinions presented are those of the committee and have not been endorsed by ASCE, SCEC, or the City or County of Los Angeles.

Appreciation is given to those who have taken their time to review this document and have provided many wise comments and suggestions: Professors Jonathan D. Bray and Raymond B. Seed of U.C. Berkeley, Professors Ellen M. Rathje and Stephen G. Wright of the University of Texas at Austin, Dr. Leland M. Kraft, Dr. Neven Matasovic, Dr. Edward Kavazanjian, Dr. Marshall Lew, Boris O. Korin, Allan E. Seward, and Larry K. Stark. Review comments were also made by John A. Barneich, S. Thomas Freeman, Yoshi Moriwaki, Sarkis V. Tatusian, and John T. Waggoner of GeoPentech and Robert A. Larson, County of Los Angeles.

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Two factors that are particularly challenging to characterize accurately are subsurface stratigraphy/geologic structure and soil shear strength. Subsurface characterization requires a thorough exploration program of borings, cone penetration tests, and/or trenches, and must identify the potentially critical soil zones. Characterization of representative soil shear strength parameters is an especially difficult step in slope stability analyses due in part to the heterogeneity and anisotropy of soil materials. Furthermore, the strength of a given soil is a function of strain rate, drainage conditions during shear, effective stresses acting on the soil prior to shear, the stress history of the soil, stress path, and any changes in water content and density that may occur over time. Due to the strong dependence of soil strength on these factors, methods of soil sampling and testing (which can potentially alter the above conditions for a tested sample relative to in-situ conditions) are of utmost importance for slope stability assessments.

This report provides guidelines on each of the above-enumerated factors, with particular emphasis on subsurface/geologic site characterization, evaluation of soil shear strength for static and seismic analysis, and seismic slope stability analysis procedures.

1.2 APPLICABLE REGULATIONS AND LAWS

The State of California currently requires analysis of the seismic stability of slopes for certain projects. Most counties and cities in southern California also require analysis of the static stability of slopes for most projects. The authority to require analysis of seismic slope stability is provided by the Seismic Hazards Mapping Act of 1990, which became California law in 1991 (Chapter 7.8, Sections 2690 et. seq., California Public Resources Code). The purpose of the Act is to protect public safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure; or other hazards caused by earthquakes. The Seismic Hazards Mapping Act is a companion and complement to the Alquist-Priolo Earthquake Fault Zoning Act, which addresses only surface fault-rupture hazards. Chapters 18 and 33 (formerly 70) of the Uniform/California Building Code provide the authority for local Building Departments to require geotechnical reports for various projects.

Special Publication 117 (SP 117), by the California Department of Conservation, Division of Mines and Geology in 1997, presents guidelines for evaluation of seismic hazards other than surface fault-rupture and for recommending mitigation measures. The guidelines in SP 117 provide, among other things, definitions, caveats, and general considerations for earthquake hazard mitigation, including seismic slope stability.

SP 117 provides a summary overview of analysis and mitigation of earthquake induced landslide hazards. The document also provides guidelines for the review of site-investigation reports by regulatory agencies who have been designated to enforce the Seismic Hazards Mapping Act.

presented in Chapter 11 represent the consensus recommendations of all practicing and academic members of the Committee (regulatory officials chose not to vote). The Committee was unable to reach consensus on acceptable seismic slope displacements, and therefore regulatory agencies will need to establish their own values for this important parameter.

The Committee actively sought input from professional and academic sources across the U.S., and this report reflects the valuable input from those individuals.

1.3 LIMITATIONS

Ground deformations under static and seismic conditions can result from a variety of sources, including shear and volumetric straining. This report focuses on slope stability and seismic slope displacements, both associated with shear deformations in the ground. Ground deformations associated with volume change, such as hydrocompression or consolidation under long-term static conditions or seismic compression during earthquakes, are not covered by the actions of this committee. In addition, ground displacements associated with post-seismic pore pressure dissipation in saturated soil, or lateral spread displacements in liquefied ground, are not covered.

The intent of this report is to present practical guidelines for static and seismic slope stability evaluations that blend state-of-the-art developments in methodologies for such analyses with the site exploration, sampling, and testing techniques that are readily available to practicing engineers in the southern California area. Accordingly, the intent is not necessarily to present the most rigorous possible procedures for testing the shear strength of soil and conducting stability evaluations, but rather to suggest incremental rational modifications to existing practice that can improve the state-of-practice. It should be noted that the Committee by no means intends to discourage the use of more sophisticated procedures, provided such procedures can be demonstrated to provide reasonable solutions consistent with then-current knowledge of the phenomena involved.

Adverse bedding conditions (out-of-slope bedding) and shear strength values representing the weaker materials (such as shale interbeds in a predominantly sandstone formation) within the mapped geologic unit are considered in the rock-strength grouping. If geotechnical shear test data are insufficient or lacking for a mapped geologic unit, the unit is grouped with lithologically and stratigraphically similar units for which shear strength data are available.

Based on calibration studies (McCrink, in press), hillslopes exposed to ground motions that exceed the yield acceleration for instability, and are associated with displacements greater than 5 cm are included in Earthquake-Induced Landslide Zones. The ground motion parameters used in the analysis include mode magnitude, mode distance, and peak acceleration for firm rock. Expected earthquake shaking is estimated by selecting representative strong-motion records, based on estimates of probabilistic ground motion parameters for levels of earthquake shaking having a 10 percent probability of being exceeded in 50 years (Petersen et al., 1996).

Seismic Hazard Zones for potential earthquake-induced landslide failure are presented on 7.5-minute quadrangle sheet maps at a scale of 1:24,000. Supplementary maps of rock strength, adverse bedding, geology, ground motions, and an evaluation report describing strength classification, Newmark displacements and regional geology and geomorphology are also provided for each quadrangle as the basis for delineation of the zones. The zone maps do not identify other earthquake-triggered slope hazards including ridge-top spreading and shattered ridges. Run-out areas of triggered landslides may extend outside the landslide zones of required investigation.

Seismic Hazard Zone maps are being released by the California Department of Conservation, Division of Mines and Geology. The maps present zones of required investigation for landslide and liquefaction hazards as determined by the criteria established by the Seismic Hazards Mapping Act Advisory Committee.

to the potential impact of the subsurface geologic structure, stratigraphy, and hydrologic conditions on the stability of the slope. The assessment of the subsurface stratigraphy and hydrologic conditions of sites underlain solely by alluvial materials may be performed by the geotechnical engineer. The shear strength and other geotechnical earth material properties should be evaluated by the geotechnical engineer. The geotechnical engineer should perform the stability calculations. The ground motion parameters for use in seismic stability analysis may be provided by either the engineering geologist or geotechnical engineer, or a registered geophysicist competent in the field of seismic hazard evaluation.

4. Presentation and analysis of the data, including an evaluation of the potential impact of geologic conditions on the project.

Geologic reports should demonstrate that each of those phases has been adequately performed and that the information obtained has been considered and logically evaluated. Minimum criteria for the performance of each phase are described and discussed below.

4.1 BACKGROUND RESEARCH

The purpose of background research is to obtain geologic information to identify potential regional geologic hazards and to assist in planning the most effective surface mapping and subsurface exploration program. The availability of published references varies depending upon the study area. Topographic maps at 1:24,000 scale are available for all of California's 7.5' quadrangles. More detailed topographic maps are often available from Cities or Counties. Most urban locations in California have been the subject of regional geologic mapping projects. Other maps that may be available include landslide maps, fault maps, depth-to-subsurface-water maps, and seismic hazard maps. Seismic slope stability hazard maps prepared by the California Division of Mines and Geology (CDMG) are particularly relevant, and the location of a site within in a seismic slope stability hazard zone will generally trigger the type of detailed site-specific analyses that are the subject of this report. The above maps are typically published by the United States Geological Survey (USGS), CDMG, Dibblee Geological Foundation, and local jurisdictional agencies (e.g., Seismic Safety elements of cities and counties). Collectively, these maps provide information useful for planning a geologic field exploration. In addition, the maps provide insight into regional geologic conditions (and possible geologic constraints) that may not be apparent from focused site studies.

Review of unpublished references also should be a part of geologic studies for slope stability. Previous geologic and geotechnical reports for the property and/or neighboring properties can provide useful data on stratigraphy, location of the groundwater table, and shear strength parameters from the local geologic formations. Strength data should be carefully reviewed for conformance with the sampling and testing standards discussed in sections 6 and 7 before being used. Critical review of topographic maps prepared in conjunction with proposed developments can reveal landforms that suggest potential slope instability. These materials are usually kept by the local jurisdictional governing agency, and review of their files is recommended.

Once review of available geologic references has been performed, aerial photographs of the area should be reviewed. Often, the study of stereoscopic aerial photographs reveals important information on historical slope performance and anomalous geomorphic features. Because of differences in vegetative cover, land use, and sun angle, the existence of landslides or areas of potential instability is sometimes visible in some photographs, but not in others. Therefore, "going into the field." The number of borings required is a function of the areal extent of the development, available information from previous investigations, and the complexity of the geologic features being investigated. Sound geologic and engineering judgment is required to estimate the number of borings required for a specific site. Guidelines on minimum level of exploration necessary for various types of construction are presented in NAVFAC 7.01 (1986). In general, it is anticipated that the number of borings/trenches should not be less than three. Additional borings will be required in many cases when the geology is complex. Borings should be positioned such that extrapolation of geologic conditions is minimized within the areas of interest.

The depth of borings and test pits should be sufficient to locate the upper and lower limits of weak zones potentially controlling slope stability. It should be noted that movement of landslides can be accommodated across multiple slip surfaces. Accordingly, locating the shallowest potential slide plane at a site may not be sufficient. In general, the depth of exploration should be sufficiently deep that the static factor of safety of a slip surface passing beneath the maximum depth of exploration and through materials for which appropriate presumptive strength values are assumed is greater than 1.5.

As noted above, continuous logging of subsurface materials is generally required to locate zones of potential weakness. Downhole logging is commonly practiced in southern California, and is widely thought to be the most reliable procedure. Downhole observation of borings provides an opportunity for direct sampling of potentially critical shear zones or weak clay seams. Such sampling and subsequent laboratory testing can be used to estimate strengths along potential slip surfaces. Prevailing conditions such as the presence of subsurface water, bad air, or caving soil may make it unsafe or impractical to enter and log exploratory borings. In those circumstances, it is necessary to utilize alternative methods such as continuously cored borings, conventional borings with continuous sampling, or geophysical techniques. Although those methodologies may be useful, the data obtained from them have limitations as geologic conditions are inferred rather than directly observed. Therefore, when such methods are utilized, the limitations should be compensated for by more subsurface exploration, more testing, more conservative data interpretation, and/or more comprehensive engineering analysis.

Detailed and complete logs of all subsurface exploration should be provided in geologic reports. Written descriptions of field observations should be accompanied by graphic logs that depict the geologic units, subsurface water conditions at the time of drilling and any subsequent measurements, and information relevant to soil sampling (e.g., sampler used, driving system, blow count, etc.) (ASTM D1586 and D6066-98).

landslide slip surfaces, and lines that represent interpretation of bedding planes, joints, or fractures. Sections that clearly show interpretation of geologic structure are necessary for subsequent engineering evaluation of stability because the ultimate determination of potential failure planes for analyses is dependent upon the accuracy of those sections. Because geologic structure is so critical to the evaluation of slope stability, potential modes of failure should be identified by the geologist, and evaluation of the most critical modes of failure should be a made by both the geologist and geotechnical engineer.

1. By the use of total unit weights and specification of groundwater table location and boundary water pressures. This method is appropriate for effective stress analyses of slope stability and should be used with effective stress strength parameters. [If a total stress analysis is desired, it should be performed with no phreatic surface (i.e., zero pore pressure). Seepage forces should not be included. Total stress strength parameters should be used.]

2. By the use of buoyant unit weights and seepage forces below the water table. This method is appropriate for use only with effective stress analyses; it should not be used with total stress analyses.

Method 1 is most commonly selected. In a stability analysis utilizing Method 1, pore-water pressures are commonly depicted as an actual or assumed phreatic surface or through the use of piezometric surfaces or heads. The phreatic surface, which is defined as the free subsurface water level, is the most common method used to specify subsurface water in computer-aided slope stability analyses. The use of piezometric surfaces or heads, which are usually calculated during a seepage or subsurface water flow analysis, is generally more accurate, but not as common. Several programs will allow multiple perched water levels to be input within specific units through the specification of piezometric surfaces.

denser, therefore, stiffer and stronger than the in-situ soil. The converse is also true, namely a dilatant sample will decrease in density as a result of the sampling process; therefore, the tested specimen will be weaker than the in-situ soil.

6.2 SELECTION OF AN APPROPRIATE SAMPLING TECHNIQUE

It follows from the above reasoning that the sampling techniques that impart the least shear strain to the soil are most desirable. Commonly available sampling techniques include: (1) driven thick-walled samplers advanced by means of hammer blows, (2) pushed thin-walled tube samplers advanced by static force, and (3) hand-carved samples obtained from a bucket-auger hole or test pit.

Two types of thick-walled driven samplers are most often used in practice: (1) Standard Penetration Test (SPT) split spoon samplers, which have a 2.0-inch outside diameter and 5/16-inch wall thickness, and (2) so-called California samplers, which typically have a 3.0- to 3.3-inch outside diameter, 1/4- to 3/8-inch wall thickness, and internal space for brass sample tubes (which typically are stacked in 1.0-inch increments).

Pushed thin-walled tube samplers are typically 3 to 5 inches in diameter with an approximately 1/16 to 1/8-inch-thick walls. When configured with a 3.0-inch outside diameter and advanced with a simple static force, they are referred to as Shelby tubes (ASTM D1587). A sampler that provides less sample disturbance than Shelby tubes is a Hydraulic Piston Sampler (e.g., Osterberg type). It is often not possible to penetrate cohesionless soil or stiff cohesive soil with Shelby tubes, and in such cases a Pitcher tube configuration can be used. The sample tube used in a Pitcher tube sampler is identical to a Shelby tube, but the tube is advanced with the combination of static force and cutting teeth around the outside tube perimeter, which descend to the base of the tube when significant resistance to penetration is encountered.

Hand-carved samples are generally retrieved by removing an intact block of soil, which is transported to the laboratory. The sample is carefully trimmed in the laboratory to the size required for testing. Disturbed bulk samples can also be hand collected for remolding in the laboratory.

The selection of a sampling method for a particular soil should take into consideration the disturbance associated with field sampling as well as transportation and laboratory sample handling. Tube samplers require specimen extrusion and trimming, whereas the brass rings used in California samplers can be directly inserted into direct shear or consolidation testing equipment.

be cleansed of contaminating materials and remolded for subsequent testing in the laboratory (see Section 7.3.3(b)ii).

5. A conservative estimate of strengths along unweathered joint surfaces in rock masses can be obtained by pre-cutting in the laboratory an intact rock specimen and shearing the sample in a direct shear device along the smooth cut surface. The strength obtained from the pre-cut sample is generally a conservative estimate because actual joint surfaces have asperities not present in the lab specimen. Alternatively the rock may be repeatedly sheared without pre-cutting the sample. The objective in sampling for this type of testing is therefore an intact rock specimen, with the "joint" surface being created parallel to the direction of testing. Such samples can be obtained by coring, hand carving, or driving samples in non-brittle rocks.

6. Intact rock should be sampled by coring or hand carving to preserve sample integrity. California samples of intact rock will generally be fractured and significantly disturbed. Accordingly, shear strengths obtained from testing of specimens obtained with California samples will generally be lower than the actual strength of the in situ intact rock.

7. For new compacted fills, bulk samples of borrow materials can be obtained for re-molding and compacting in the laboratory.

8. Soil containing significant gravel generally can be sampled by hand carving of large specimens or correlations with penetration resistance can be used to estimate strengths. Correlations with penetration resistance are based on SPT blow counts or Becker penetrometer blow counts. Andrus and Youd (1987) describe a procedure to determine N-values in soil deposits containing significant gravel fragments. They suggest that the penetration per blow be determined and the cumulative penetration versus blow count be plotted. Changes in the slope of the plot indicate that gravel particles interfered with sampler penetration. Estimates of the effective penetration resistance of the soil matrix can be made for zones where the gravel particles did not influence the penetration.

6.3 SPACING OF SAMPLES

For most projects, samples from borings should be obtained at maximum 5-foot vertical intervals or at major changes in material types (whichever occurs more frequently). Samples in heterogeneous or layered materials should be obtained as often as needed to reflect the variability of the deposit and retrieve samples of the weakest materials that might influence slope stability. Larger sample-spacing intervals can be used for deep borings drilled primarily to obtain information on geologic structure

Table 7.1. Summary of Recommended Strength Evaluation Procedures

Site Condition	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Fine-grained soft alluvium loaded by fill	Undrained	Total	Peak	Reduce peak strength by 30%	UTC (UU or CU) Vane Shear	Undrained, total stress, UTC (UU or CU), use judgment for pk. v. residual
Coarse-grained alluvium loaded or unloaded (unsaturated)	Drained	Effective	Peak	None	DDS, DTC	Effective Stress, drained, DDS, DTC
Coarse-grained alluvium, loaded or unloaded (saturated)	Drained	Effective	Peak	Check for liquefaction potential	DDS, DTC	Effective Stress, drained, DDS, DTC; use undrained residual strength if liquefiable
Saturated, fine-grained, overconsolidated, stiff alluvium or clayey bedrock with massive or supported bedding, Loaded	Undrained (check drained)	Total	Peak	Reduce peak strength by 30%	UTC	Undrained, total stress parameters, rate adjusted peak strengths
Unloaded	Drained	Effective	Depends on LL and CF	None	DDS, DTC (see Comment 3)	
Heavily overconsolidated saturated clay or clayey bedrock - pre-existing shear surfaces, loaded or unloaded	Drained	Effective	Residual	None	DDS, RS	Effective Stress, Drained DDS, RS

For the rapid stress application that occurs during earthquake shaking, shearing occurs under undrained conditions. For that condition, the following types of strength parameters are recommended:

- Clay: Total-stress strength parameters from undrained test (CU or UU)
- Clay at residual: Effective-stress strength parameters, drained or undrained test
- Sand, unsaturated: Effective-stress drained strength parameters
- Sand, saturated: See below

For saturated sands, the pore pressure generated during shaking should be estimated with a liquefaction analysis. The undrained residual strength should be used if the soil liquefies, which can be estimated using available correlations with penetration resistance (i.e., Fig. 7.7 of Martin and Lew, 1999). A drained strength should be used if the soil does not liquefy, but the pore pressure generated during shaking should be estimated, so that the effective stress in the soil can be appropriately reduced.

The criteria in the "Seismic" column of Table 7.1 can be applied to the selection of strengths for seismic stability analyses. The principal comments associated with those criteria are as follows:

With respect to strain-softening effects, initial analyses can be performed with peak strengths. However, if slope displacement analyses indicate significant shear deformations in the slope, strengths should be reduced to values between peak and residual (depending on the soil characteristics and the amount of the computed displacement).

As discussed in Section 7.2.4, rate effects tend to increase the undrained strength of fine-grained materials, but may be partially offset by cyclic strength degradation effects.

7.2 GENERAL CONSIDERATIONS

7.2.1 Drainage Conditions and Total vs. Effective Stress Analysis

Soil behavior during drained loading is fundamentally different than during undrained loading. Drained loading implies that loads are applied at a sufficiently slow rate that no pore pressures are generated in the soil during shear, and volume change is allowed. Brinch-Hansen (1962) referred to this as "consolidated-drained" or CD loading, and that nomenclature will be used here. Undrained loading refers to a shear condition in which no volume change occurs, accordingly increased pore pressures will be generated in saturated, contractive soil, and decreased pressures in saturated, dilatent soil. Undrained shear can occur immediately after construction, or upon loading that follows consolidation of the soil. These cases are referred to

The undrained shear strength of soil also can be described using effective stress strength parameters, but this is seldom done in routine practice because the use of such parameters in design would require an evaluation of pore-pressure response in the field during construction, which is a non-trivial analysis. Accordingly, shear strengths from UU or CU tests are typically defined using alternative strength parameters. End-of-construction (UU) strengths are described using conventional total stress strength parameters, i.e.,

$$\tau_{ff} = c + \sigma_{f,f} \tan \phi \text{ (end-of-construction, UU)} \qquad (7.1b)$$

where σ_{ff} = total normal stress on the failure plane at failure. This linear approximation is only appropriate over a fairly short range of normal stresses. For saturated soil, ϕ=0 in Eq. 7.1b, and the strength is often denoted as $\tau_{ff} = s_u$ or $\tau_{ff} = c$. As illustrated in Fig. 7.2, these strength parameters are generally obtained with triaxial testing, as sample drainage cannot readily be controlled in direct shear tests. As indicated in the figure, triaxial tests are performed at a cell pressure σ_{cell}, and the shear strength τ_{ff} is obtained as half the deviatoric stress ($2q_f$).

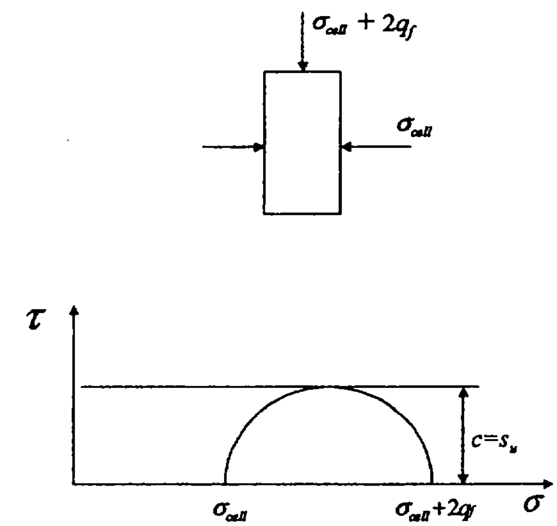


Figure 7.2. Stress State at Failure in Triaxial UU Test

As described by Casagrande and Wilson (1960) and Ladd (1991), post-consolidation, undrained (CU) strengths are evaluated by first consolidating the soil to a specified effective consolidation stress, σ_c', and then shearing the soil rapidly to failure. The shear stress on the failure plane at failure (τ_{ff}) is best evaluated by plotting the Mohr Circle in effective stress space, as shown

5. Unloading of soft clay may be critical under short-term undrained or long-term drained conditions. Strengths representative of both conditions should be evaluated for stability analyses.

For saturated or nearly saturated soils, rapid stress application during earthquake shaking occurs as undrained loading. Accordingly, either total stress or CU strength parameters should be used. If, prior to the probable earthquake, effective stresses in the soil can be expected to change with time due to consolidation, it may be reasonable to use CU strengths based on effective consolidation stresses that will be present in the slope after the completion of some acceptable amount of consolidation. Assuming the construction being analyzed involves loading of the ground, the range of effective possible consolidation stresses that could be chosen is, as a minimum, the effective consolidation stress prior to construction, and as a maximum, the effective consolidation stress after all excess pore pressures from loading have dissipated. The choice of which consolidation stress within this range should be used is project-specific, and should be selected after discussion between the consultant and regulatory official. Conversely, clayey soil subject to unloading will swell over time, and the reduced effective stresses present after the completion of swell should be used for seismic design.

Negative pore pressures are present in unsaturated soils. Limited experimental and centrifuge studies have shown that at saturation levels of 88% and 44%, these negative pore pressures may rise (i.e., become less negative) during rapid cyclic loading (Sachin and Muraleetharan, 1998; Muraleetharan and Wei, 2000). The available information is far from exhaustive, but those studies preliminarily suggest that at the pre-shaking saturation levels considered, the pore pressures can rise to nearly zero, but are unlikely to become positive. That behavior is less likely to occur in materials with higher degrees-of-saturation (for example, > 90%), because the relative scarcity of air bubbles could lead to the development of positive pore pressures. Accordingly, for materials that can be expected to have moderate saturation levels (< 90%), an assumption of zero pore pressure in the soil is likely to be conservative, meaning that stability analyses can be performed using effective stress strength parameters derived from drained shear tests. Those strength parameters should be used with effective stresses calculated for a zero pore pressure condition (i.e., effective stress = total stress).

7.2.2 Post-Peak Reductions in Shear Strength

All limit equilibrium methods for slope stability assume a rigid-perfectly plastic soil stress-deformation response, as depicted in Fig. 7.3. Because this model assumes strength to be independent of deformation, it can be difficult to apply to soil subject to post-peak reductions in shear capacity (i.e., soil with strength that is dependent on the level of deformation). Many soils strength is measured (i.e., intact specimen for ultimate; reconstituted specimen for fully softened).

The above strength terms are used in the context of drained shear. Undrained specimens can also experience strain softening, often due to pore pressure increase and/or particle re-orientation. For undrained shear, we will only refer to two strength values - peak and residual.

Skempton (1985) reports that fully softened/ultimate and residual drained shear strengths are approximately equivalent for materials with clay contents less than 25% (with clay defined as material finer that 0.002 mm). Drained residual strengths are less than fully softened strengths for materials with higher clay contents.

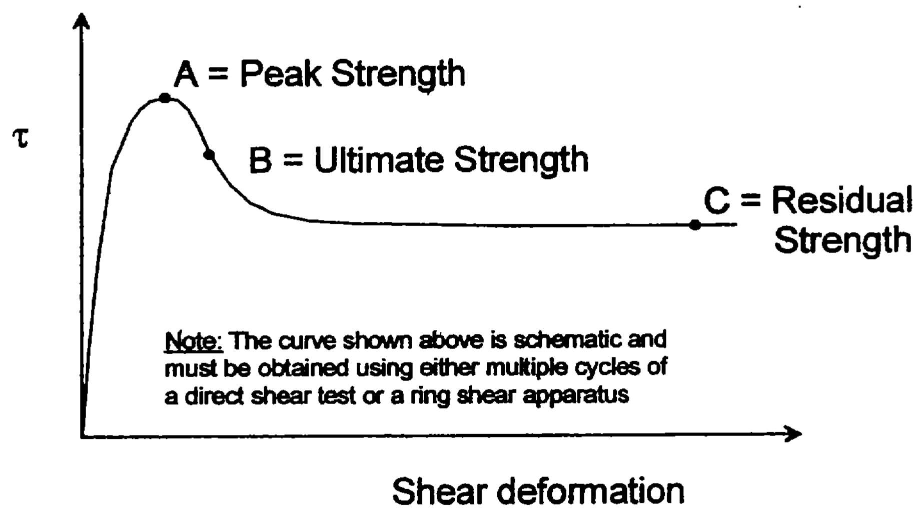


Figure 7.4. Diagrammatic Stress-Displacement Curve

Many materials can experience a post-peak reduction in strength, including most clayey soil (under drained or undrained conditions), dense sand under drained conditions, loose sand under undrained conditions, and cemented soil.

The following guidelines apply to the selection of appropriate strength parameters in materials subject to strain softening during long-term, drained loading conditions.

1. Residual strengths should be used in materials that have experienced significant previous shear deformations. Examples include materials located along pre-existing landslide slip surfaces and along continuous bedding planes likely to have been subject to significant past movement (e.g., folded bedrock that may have experienced flexural slip along bedding planes). Residual strengths should be used in those materials, even if the relative movement across the discontinuity occurred thousands of years ago (Skempton and Petley, 1967).

slope failure mechanisms at the site, and strain compatibility of shear strengths for materials along the failure surface.

Recommendations 3, 5, and 6 above are based on comparisons of mobilized shear strength (established from back analyses of first time slides) to fully softened and residual shear strengths by Stark and Eid (1997), and updated by Stark and McCone (2001). The Committee recognizes that ground conditions at the sites considered by Stark and Eid (1997) may not be directly comparable to materials that weather from older bedrock (pre-Quaternary). It is, however, the consensus of the Committee that these recommendations represent the best approach currently available. With respect to Recommendation 4 (weathered soil), the samples tested for Atterberg limits and shear strength should be taken from naturally weathered deposits of a similar earth material at or near the site. To distinguish between the levels of plasticity referred to above, visual classifications can be used in lieu of formal Atterberg Limits testing.

For undrained loading of clayey soil, Ladd (1991) found back-calculated values of tan(Ψ_u) from field case histories to be similar to laboratory CU test results adjusted for strain compatibility effects. The laboratory CU parameters for which these comparison were made represent peak strengths, hence, it is inferred that strain-compatibility adjusted peak strengths can be used for field applications. Strain compatibility adjustments to peak shear strength are discussed in Section 4.9 of Ladd (1991).

7.2.3 Soil Anisotropy

Stress and fabric induced anisotropy, as well as pre-existing shear zones, can lead to shear strengths that are dependent on the orientation of the failure plane. Slopes with pre-existing shear zones should be analyzed using along-bedding and across-bedding strengths applied to relevant portions of the failure surface (guideline #4 for sampling along bedding is included in Section 6.2).

For relatively homogeneous alluvial soil subjected to undrained loading, laboratory testing that shears samples across horizontal planes (such as triaxial tests on specimens retrieved from vertically advanced samplers) generally provide unconservatively high estimates of shear strength along the actual failure surface in the field (Duncan and Seed, 1966a and 1966b). Such effects are less significant for homogenous soil subjected to drained loading (Mitchell, 1993).

7.2.4 Rate Effects

Laboratory shear tests are generally performed over the course of minutes to days. Field loading under static loading is much slower, whereas seismic loading is more rapid.

strain rates can be used as a first-order approximation of the residual strength friction angle under undrained and rapid loading conditions.

7.2.5 Effect of Confining Stress on Soil Failure Envelope

The effect of confining stress on the stress-strain response of granular materials has been summarized by Lambe and Whitman (1969) as follows:

1. As confining pressure increases, the peak normalized shear strength (i.e., secant friction angle based on peak strength) decreases.

2. The fully softened/ultimate strength is more-or-less independent of changes in confining pressure.

The strong effect of confining pressure on normalized peak shear strengths has been attributed to a decreased tendency for dilation at large confining pressures, and a reduced level of grain interlocking (and increased grain crushing) as confining pressures increase (Lambe and Whitman, 1969; Terzaghi et al., 1996). This reduction of friction angle with increasing confining pressure causes downward curvature of the failure envelope.

For clayey soil, Skempton (1985) and Stark and Eid (1994) have found downward curvature of failure envelopes representing the residual strengths, and Stark and Eid (1997) have found downward curvature of failure envelopes for fully softened strength. Therefore, curvature of failure envelopes is an issue faced in both cohesive and cohesionless materials. At low confining pressures, curvature can be particularly pronounced, as failure envelopes for residual strength pass through or nearly through the origin

Given the above, it is important to perform shear strength testing across the range of normal stresses expected in the field. A curved representation of the failure envelope can be used in many modern computer programs, and is the preferred method for accounting for these effects. If this is not possible, a linear representation of the actual curved failure envelope can be used across the range of normal pressures expected in the field. It should be noted, however, that, in situations where both shallow and deep-seated stability must both be analyzed, more than one linear envelope would need to be established.

At sites with particularly deep-seated slip surfaces, it may not be possible to perform testing at the normal pressures occurring in the field. In such cases, testing should be performed across a range of lower normal stresses to establish the variation of friction angle with increased stress. This variation can be described in terms of power, cycloid, and hyperbolic equations (Duncan et al., 1989; Atkinson and Farrar, 1985; Maksimovic, 1989; Vyalov, 1986). These expressions can

7.3.1 Presumptive Values

Conservative presumptive shear strength parameters can be used in slope stability analyses for sites where no field exploration or laboratory testing have been performed. Because these presumptive strength parameters are used in lieu of site-specific exploration or testing, they must be chosen conservatively, so that the probability that lower strength parameters exist at a site is very low. In general, presumptive values should be selected and approved by local regulatory reviewing agencies in a manner that incorporates data from local case histories, experimental data, and back analyses. These values apply only for the drainage conditions, loading rates, etc. that were present in the tests/case studies from which the values were derived. Provided they are used for a comparable set of conditions, presumptive strength parameters should yield a safe design, but not necessarily an economical one. For most projects, it should be economically beneficial to perform field exploration and laboratory testing to develop project-specific shear strength parameters rather than use low, presumptive strength values. It also should be noted that presumptive strength parameters are intended to be realistic lower bound strength values and are not intended to be lower than any values ever obtained.

7.3.2 Published Correlations

As described previously in Section 6.2, in most cases the drained strength of sand and non-plastic silt is best estimated by correlations with SPT blow count and CPT tip resistance. The recommended SPT correlation for sand is shown in Fig. 7.5a. Note that the blow count $[(N_1)_{60}]$ is corrected for procedure to 60% efficiency, and corrected to 1.0 atm overburden pressure. CPT tip resistance is also normalized to 1.0 atm overburden pressure in the correlation shown in Fig. 7.5b. SPT and CPT procedure and overburden correction factors are discussed in detail in Martin and Lew (1999).

Evaluation of the drained or undrained shear strength of clay should be accomplished with testing. However, it is good practice to check laboratory-derived strength parameters for clay using available correlations. A particularly onerous problem with clay strength evaluations can be the evaluation of residual shear strengths for thin failure surfaces. This problem arises principally from difficulty in sampling and properly orienting test specimens in direct shear devices. Accordingly, it is strongly recommended that sufficient clay be obtained by scraping the surface to allow determination of the liquid limit and clay fraction, so that the residual shear strengths for clay slip-surfaces can be checked using published correlations such as those by Stark and McCone, 2001 (updated from Stark and Eid, 1994 and 1997). Correlations between soil liquid limit and clay fraction (established by a ball-milling technique) and friction angle are shown in Figures 7.5c (residual friction angle) and 7.5d (fully softened friction angle). Care should be exercised when using these correlations because liquid limits and clay contents derived

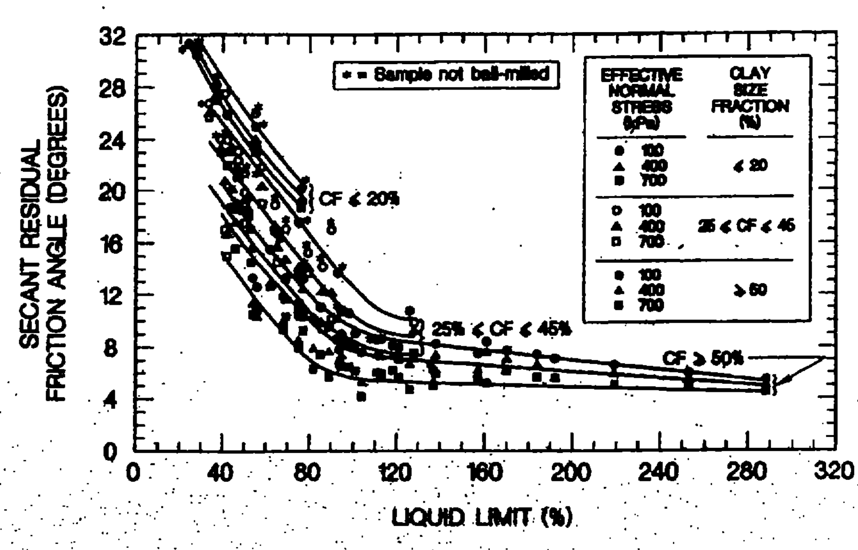


Figure 7.5c. Empirical Correlation Between Drained Residual Friction Angle of Fine-Grained Soil and Ball-Milled Liquid Limit (Stark and McCone, 2001)

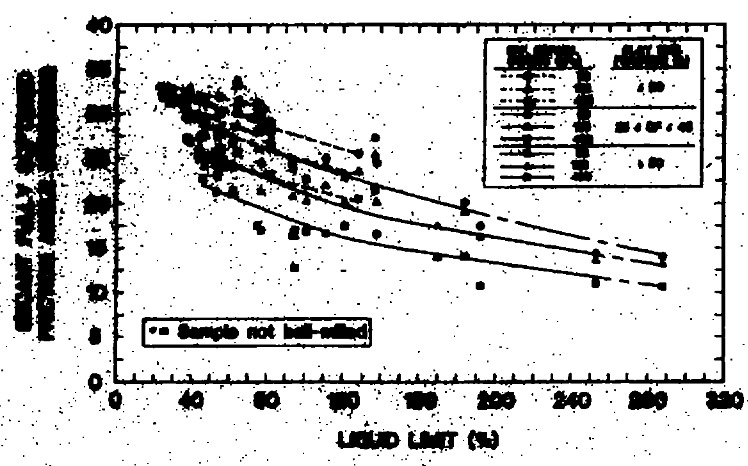


Figure 7.5d. Empirical Correlation Between Fully Softened Friction Angle of Fine-Grained Soil and Ball-Milled Liquid Limit (Stark and McCone, 2001)

7.3.3 Laboratory Testing

(a) General Considerations

Laboratory testing can be used to evaluate the load-deformation response and shear strength of soil samples. Laboratory equipment available for shear-strength testing includes the following:

- The triaxial compression test (TC) is a relatively common laboratory test that can be used for the evaluation of drained or undrained shear strength parameters. The applied load is measured in terms of deviatoric stresses, and deformation is measured in terms of axial strains.

- Unconfined compression tests are simply UU triaxial compression tests with zero cell pressure. Unconfined compression tests are only useful for crude estimation of total stress strength parameters, and tend to provide conservative results. These strengths can generally be applied only for an "unconsolidated" condition (i.e., no field consolidation since sample retrieval), and only for the location in the ground from which the sample was retrieved.

- The direct shear test (DS) is the most commonly used shear strength test due to its operational simplicity. In southern California, the test is often run on specimens retrieved from California samplers, which (as noted in Section 6.2) are likely to be significantly disturbed. DS test results for such specimens are very approximate. In the DS test, applied load is measured in terms of shear stress, and deformation is measured in terms of shear displacement (not strain). The ASTM procedure for this test is formulated to achieve drained shear. True undrained conditions cannot be obtained because pore pressures dissipate during shear. The direct shear test controls the location of shearing and is therefore useful for testing specific failure surfaces. DS testing devices can be used to subject a sample to multiple cycles of shearing, which allows an estimation of residual strength. Unfortunately, the results may be unconservative (Watry and Lade, 2000), and should always be checked against either correlations (Stark & McCone, 2001) or results of ring shear testing (discussed below).

- Ring shear tests can be used to estimate the residual strengths corresponding to large displacements in reconstituted (bulk) samples. Ring shear devices cannot be used with undisturbed soil specimens from the sampler types discussed in Section 6.0.

- Although mostly research tools at this point, direct simple shear and torsional shear testing provides a reliable means of evaluating either undrained or drained stress-strain response of soil.

endorse such practice. Furthermore, the absence of an ASTM standard for that test makes it a non-standard test that in practice will vary in procedure and quality from consultant to consultant, and one that has not benefited from a comprehensive review and comparison with truly undrained tests. Although this committee cannot endorse such a practice, some Committee members believe that the appropriate regulatory agencies have the power to decide under which testing conditions (if any) rapid, so-called "undrained" direct shear tests can be used to estimate undrained strength parameters in their individual jurisdictions. Other Committee members believe that the use of rapid deformation rates in the direct shear test device (in an effort to approximate undrained strength parameters) should not be allowed at this time, because it can lead to unreasonable and unconservative estimates of the undrained shear strength.

The following guidelines should be adhered to so that the test results can be used for slope stability analyses.

1. The dry density and moisture content prior to shear should be determined. That can be achieved by measuring the weight of the ring sample prior to testing and determining the moisture content using an adjacent ring.

2. Samples tested for static stability analyses should be saturated unless the engineer can convincingly demonstrate that saturation of the soil during the design life of the slope is unlikely. Samples tested for seismic stability analyses may be tested at field moisture conditions that are likely to exist at the time of the earthquake. For non-irrigated slopes, that may be the long-term average field moisture condition. For irrigated slopes, samples should be tested under saturated conditions. It should be noted that soaking a sample from both top and bottom can result in trapped air inside of the sample. It is often advantageous to soak samples only from the bottom until the surface of the sample suggests that soaking has achieved saturation by capillary rise.

3. Normal stresses need to be consistent with the problem being analyzed. For example, to analyze the surficial stability of a slope requires knowledge of the shear strength at normal stresses on the order of only 200 psf, which requires testing at very low confining stresses.

4. In order to obtain drained strength parameters, the speed of the direct shear test needs to be slow enough to ensure that pore pressures dissipate inside the sample. According to ASTM, the maximum speed is a function of t_{50}, which can be determined from consolidation theory using the Casagrande or Taylor methods (e.g., Holtz and Kovacs, 1981). Currently, ASTM D-3080 specifies that the time to failure is to be greater than $50 \cdot t_{50}$. Table 7.3 provides guidelines to assist in the specification of deformation rate for a direct shear test. These are based on correlations between coefficient of consolidation (c_v) and liquid limit from the U.S.

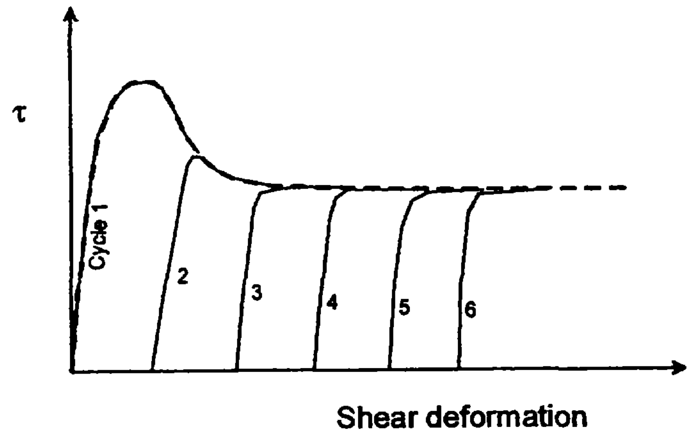


Figure 7.6. Schematic of Multiple-Cycle Direct Shear Test Results

Table 7.3. Reference Values of Time-to-Failure in Drained Direct Shear Test

Liquid Limit	Sample Condition	Time to Failure (hrs)*
40	Over Consolidated	0.25
	Normally Consolidated	1.5
	Remolded	6.0
60	Over Consolidated	1.5
	Normally Consolidated	4.0
	Remolded	15.0
80	Over Consolidated	4.0
	Normally Consolidated	10.0
	Remolded	30.0

* assuming 1.0 inch sample height and double drainage (multiply recommended times by 4.0 if drainage is only provided on one side of sample).

ii. Remolded Samples

<u>Direct shear testing is often performed on remolded samples to evaluate either fully softened or residual strengths</u>. Remolded samples should be prepared to approximate either the existing or the most critical anticipated conditions. The soil moisture content and density must both be carefully selected and controlled to achieve a sample that will yield a representative shear strength. The Committee recommends that samples that will be tested with a direct shear apparatus be remolded using the following guidelines. A bulk sample of the soil should be moisture conditioned to a moisture content at or above the optimum moisture content as

unconsolidated undrained test (UU), in which drainage is not permitted during the application of confining pressure or shear.

As described in Table 7.2, CU or UU tests are recommended to determine the undrained shear strength of soft clay under static loading. In addition, CD tests are recommended together with the drained direct shear test to determine drained strengths of sand, very stiff clay, and clayey bedrock. The following additional discussion and guidelines are provided in this section with regard to the use of CU and CD tests for slope stability problems: CU tests should be performed in accordance with ASTM D4767-95, UU tests in accordance with ASTM D2850-95 (1999), and CD test in accordance with U.S. Army Corps of Engineers EM1110-2-1906.

In piston-type test equipment (in which the axial loads are measured outside the triaxial chamber), piston friction can have a significant effect on the indicated applied load, and measures should be taken to reduce the friction to tolerable limits.

The specimen cap and base should be constructed of lightweight material and should be of the same diameter as the test specimen in order to avoid entrapment of air at the contact faces.

The porous stones should be more pervious than the soil being tested to permit effective drainage.

Rubber membranes used to encase the specimen should provide reliable protection against leakage, yet offer minimum restraint to the specimen. Commercially available rubber membranes having thicknesses ranging from 0.0025 in. (for soft clay) to 0.01 in. (for sand or clay containing sharp particles) are generally satisfactory for sample diameters less than 2.5 inches. Rubber membranes about 0.01 in. or greater in thickness are suitable for larger specimens.

The sample specimen height-to-diameter ratio should be between 2 and 2.5. The largest particle size should be smaller than 1/6 the specimen diameter. If, after completion of a test, it is found based on visual observation that oversize particles are present, that information needs to be included in the report.

The average height of the specimen should be determined from at least four measurements, while the average diameter should be determined from measurements at the top, center, and bottom of the specimen as follows:

$$D_{avg} = \frac{D_{top} + 2D_{center} + D_{bottom}}{4} \tag{7.2}$$

For CU tests, failure can be defined either as the maximum deviator stress $(\sigma_1'-\sigma_3')_f$, the maximum obliquity, $(\sigma_1'/\sigma_3')_f$, or the stress at a certain specified axial strain. For dilative samples, a maximum deviator stress criteria may not be determined as its value will continue to increase with deformation. However, maximum obliquity value will reach a maximum and will not increase with the deformation. Therefore, for contractive samples, maximum obliquity criteria should be used for defining the failure. For dilative samples, either maximum deviator stress or maximum obliquity criteria will provide the same measure of shear strength; however, typically the maximum deviator stress is used in slope stability

(d) Laboratory Test Data Interpretation

The number of tests needed to estimate the shear strength of a geologic unit depends on factors such as local experience with the material, continuity of strata, spatial variability of properties, and consequences of erroneous estimation. When the number of tests performed is limited, appropriate conservatism should be used to select shear-strength values for slope stability analysis. The following general guidelines should be considered when testing shear-strength samples, and analyzing and applying their results.

If data are being developed to estimate the shear strength of a relatively homogeneous deposit (such as a uniform natural deposit or an artificial fill), a sufficient number of tests should be performed to characterize the variation that is likely to result from the natural process or construction techniques, considering the materials that are available to form the deposit. The results from a number of tests can be averaged, provided they are weighted in proportion to their abundance in the slope being analyzed. Alternatively, each layer could be entered into the slope stability analysis. If a wide variation in shear strength is observed across a large project site, it is necessary to verify that the strengths used for analysis of a specific slope are representative of the materials at that location.

If data are being developed to estimate the across-bedding strength of a layered deposit, the tests should be performed on representative material samples from each of the types of layers present. In many cases, an approximately weighted average value of shear strength can be used to model the across-bedding strength. Summary plots of shear strength data for each type of material in the layered deposit should be prepared. The test results from each type of material in a layered deposit should be averaged first. Then those averaged results should be weighted in proportion to their abundance and combined with similar results from other layers to obtain an overall weighted average. The engineer should be sure to consider the possibility that large-scale properties such as variations in cementation and fracturing could affect the strength of the deposit in a manner that might not be adequately represented by the laboratory test results.

The relation between the correction factor, μ, and the plasticity index, PI, has been obtained from field case history data and is shown in Figure 7.7.

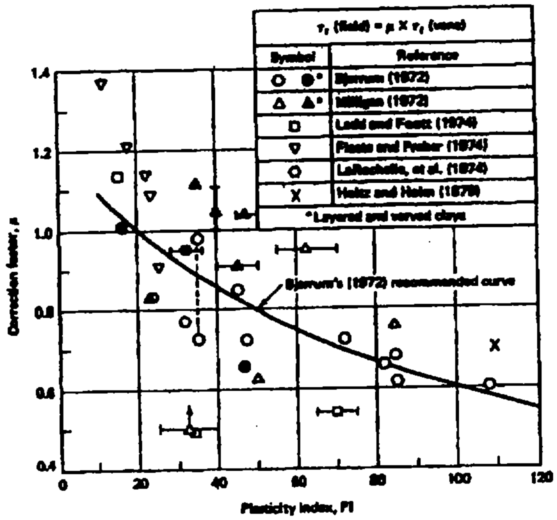


Fig. 7.7. Correlation Factor for the Field Vane Test as a Function of PI, Based on Embankment Failures (from Holtz and Kovacs, 1981)

7.3.5 Back Calculation of Strength Along a Failure Surface

Existing landslides offer the opportunity to estimate the average shear strength properties along the failure surface by mathematical methods. This procedure is generally referred to as back calculation or back analysis. The procedure requires the determination of the configuration of the landslide failure surface relative to the topography at the time of failure, variability in earth materials along the failure surface, the subsurface water level at the time of failure, external loading conditions, and the appropriate soil density. Once the above information is known, a mathematical analysis method appropriate to the slide configuration is chosen. The data described above are input into the analysis method, and an initial estimate is made of the shear strengths along the failure surface. The shear strength parameters are then adjusted and the analysis repeated until a factor of safety of 1.0 (FS=1.0) is obtained. This method provides different sets of cohesion, c, and friction angle, ϕ, which satisfy FS = 1.0. The engineer then selects an appropriate combination of c and ϕ. These strength parameters can then be utilized in the evaluation of alternate repair procedures. Skempton (1985) compared drained shear strengths obtained by careful testing of high-quality slip-surface samples with strengths determined by back calculation of the slides and found good correlation, indicating that the back-calculation method is valid for drained failures.

8 SOIL UNIT WEIGHT

The soil unit weight is required for the analysis of slope stability. The added weight due to the presence of subsurface water is accounted for by using the saturated unit weight of the soil. The use of the saturated unit weight (γ_{sat}) of the soil is conservative for most analyses. Although variations in moisture content (varying from dry to saturated) are possible, slope stability analyses should be performed using the saturated unit weight (unless specific justification for doing otherwise is provided by the consultant and approved by the regulatory reviewer). The estimation of saturated soil unit weight can be evaluated from the dry unit weight (γ_d) as follows,

$$\gamma_{sat} = \gamma_w + \gamma_d \left(\frac{G_s - 1}{G_s} \right) \tag{8.1}$$

where G_s = specific gravity of solids (typically 2.65-2.75),
γ_w = unit weight of water (62.4 pcf for fresh water)

In addition, relatively small (5 to 10 pcf) changes in density typically have little influence on the results of slope stability analyses. Saturated unit weights should be obtained from laboratory moisture-density tests on driven samples or conservative estimates from published sources such as the Slope Stability Reference Guide for National Forests in the United States (Hall et al., 1994).

mathematical models for slope stability calculations and the ability of the analyst to find the critical failure surface geometry.

Historically, the most commonly required factors of safety in southern California have been 1.5 for static long-term slope stability and 1.25 for static short-term (during construction) stability. Those factors of safety were established when computations were performed with slide-rules, when analysis methods solved at best two conditions of equilibrium, when only a few potential failure surfaces were analyzed, and when our understanding of factors influencing the shear strength of soil was less advanced. The level of uncertainty associated with those analyses justified the use of relatively high factors of safety.

The availability and speed of personal computers has allowed the development of more precise methods of analysis, which satisfy all three equations of static equilibrium, and the analysis of hundreds to thousands of potential failure surfaces. Therefore, the uncertainty related to computational methods and determination of the critical failure surface has been significantly reduced in recent years. Accurate representation of the soil shear strength for the problem being solved therefore introduces the highest level of uncertainty into current analyses. The Committee believes that the current static factors of safety remain applicable in cases where the shear strength of soil is determined by limited laboratory testing or by the use of the median values from standard correlations. However, we also believe that consideration should be given in the future to the use of lower factors of safety when uncertainty related to the shear strength is relatively small. For example, uncertainty is reduced when the shear strength is determined by back analysis of a well documented slope failure (in terms of geometry and water conditions). The Committee is not prepared to recommend specific lower safety factors at this time, but believes that this topic deserves consideration by controlling agencies.

The use of a factor of safety greater than 1.5 for static analyses is recommended if a slope in fractured or jointed cemented bedrock is analyzed using peak strength parameters derived from high quality samples of unfractured material. The use of a higher factor of safety is suggested in this instance because the joints and fractures introduce random planes of weakness into the deposit, which can significantly reduce the overall shear strength of the deposit. It is the Committee's judgment that factors of safety as high as 2.0 should be considered when a cemented material exhibits significant post-peak strength loss and contains a significant number of fractures in the location being analyzed. It should be noted that this higher factor of safety is not intended to be used when shear strengths are evaluated from de-aggregated samples.

analysis as a whole, which is most significantly influenced by the uncertainty in input parameters (such as soil strength). However, in situations where good quality sampling and testing have revealed consistent strength parameters or where regional knowledge dictates the use of specific parameters, the method of analysis can significantly affect the calculated FS.

The methods of Morgenstern and Price, Spencer, Sarma, Taylor, and Janbu's generalized procedure of slices satisfy all conditions of equilibrium and involve reasonable assumptions. Bishop's modified method does not satisfy all conditions of equilibrium, but is as accurate as methods that do, provided it is used only for circular surfaces. Duncan (1996) has found all of these methods to provide answers within 5% of each other.

Table 9.1. Characteristics of Commonly Used Methods of Limit Equilibrium Analysis (after Duncan, 1996)

Friction Circle Method (Taylor)	1937	Moment and force Equilibrium	Circular	Resultant tangent to friction circle
Ordinary Method of Slices (Fellenius)	1927	Moment Equilibrium of entire mass	Circular	Normal force on base of slice is W cos α and shear force is W sin α
Method of Slices (Fellenius)	1910	Force equilibrium of each slice		No interslice forces
Bishop's Modified Method	1955	Vertical equilibrium and overall moment equilibrium	Circular	Side forces are horizontal
Janbu's Simplified	1968	Force equilibrium	Any shape	Side forces are horizontal
Modified Swedish Method (U.S. Army Corps of Engineers Method)	1970	Force equilibrium	Any shape	Side force inclinations are equal to the parallel to the slope
Lowe and Karafiath's Method	1960	Vertical and horizontal force equilibrium	Any shape	Side force inclinations are average of slope surface and slip surface (varies from slice to slice)
Janbu's Generalized Method	1968	All conditions of equilibrium	Any shape	Assumes heights of side forces above the base vary from slice to slice
Spencer's Method	1967	All conditions of equilibrium	Any shape	Inclinations of side forces are the same for every slice; side force inclination is calculated in the process of the solution
Morgenstern and Price's Method	1965	All conditions of equilibrium	Any shape	Inclinations of side forces follow a prescribed pattern; side forces can vary from slice to slice
Sarma's Method	1973	All conditions of equilibrium	Any shape	Magnitudes of vertical side forces follow prescribed patterns

9.1e-f). In general, failure geometries with a near 90-degree angle in the lower portion of the slope should be avoided as these geometries will lead to unreasonable high normal stress concentrations near the right angle bend in the failure surface.

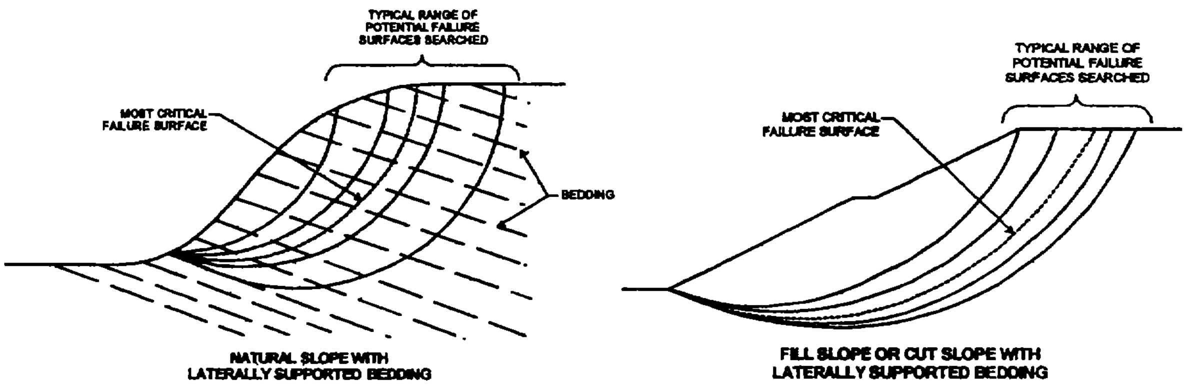


Figure 9.1a - b. Examples of Use of Circular Failure Surface Geometry

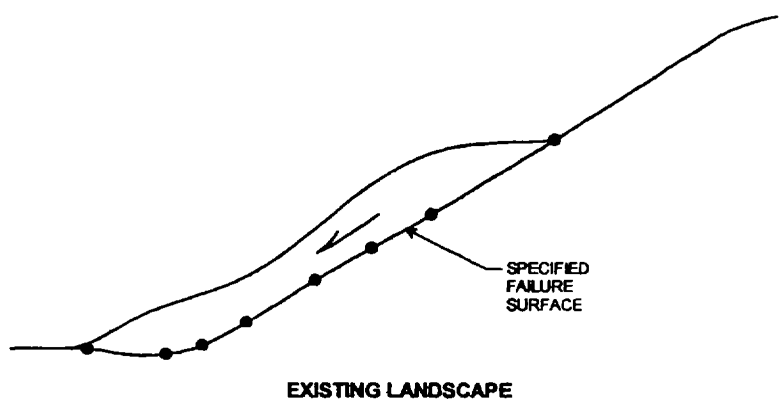
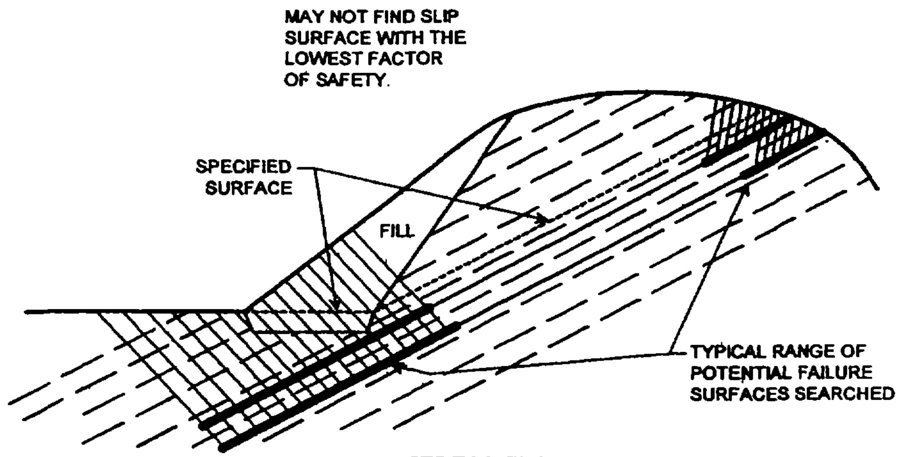
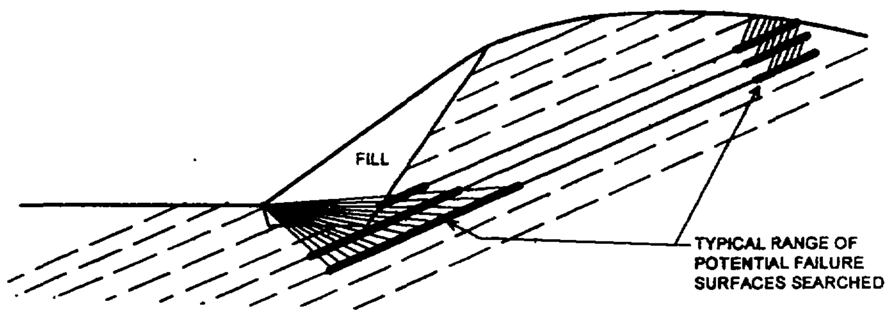


Figure 9.1c. Example of Use of Specified Failure Surface Geometry for Existing Landslide



BUTTRESS FILL



BUTTRESS FILL

Figure 9.1f. Failure Surfaces Combining Along-Bedding and Cross-Bedding Failure - Buttress Fill (bottom diagram indicates correct geometries)

9.3.2 Tension Cracks

Tension cracks or vertical fractures may form at the crest of a slope or near the head of a landslide as failure is approached. Tension cracks should be considered in slope stability calculations, and in some cases those cracks should be assumed to have water in them. The tension crack lateral location along the slope should be the one that produces the lowest factor of safety, but in practice it may not be necessary to expend the iterative effort needed to determine the most critical position.

For most situations, the approximate depth of the tension crack can be estimated from the following equations. If the material through which the crack will form is generally homogeneous and isotropic, the depth of the tension crack may be estimated from:

local minimums are found. If the computer program works by generating a large number of circular surfaces in a random manner, the engineer needs to direct the computer to search enough surfaces so that adding more surfaces does not result in a significantly lower factor of safety.

If non-circular failure surfaces are to be used, geologic judgment and kinematics need to be considered. For example, if Spencer's method is used to generate a failure surface that has a nearly right-angle bend (see Figure 9.1e-f, upper frames) a kinematically unreasonable geometry results and the calculated factor of safety may be too high. That problem can be detected by checking for very high base-of-slice normal-stresses and shear resistances in narrow slices. Those high stresses and resistances result from the concentration of high side forces at the right-angle bend, which creates high base-of-slice normal-forces and unreasonably high shear-resistance. Spencer's analysis can yield factors of safety that are significantly higher than those produced by a simplified Janbu analysis when kinematically unreasonable surfaces are specified (dip-slope analyses with passive toe wedges can create that problem). The problem can often be resolved by searching for similar, but kinematically more reasonable surfaces, in nearly the same area (see Figure 9.1e-f, lower frames). If a computer program is used to generate a large number of non-circular randomly shaped surfaces, the engineer should carefully evaluate the results for convergence, since good geotechnical and geologic judgment can often result in finding more critical failure surfaces. To provide some guidance, several examples of procedures that can be used to search for the critical failure surface are shown on Figure 9.1

9.3.4 Search for Critical Failure Direction

Existing or potential failures that do not occur directly downslope require consideration of the critical direction of analysis (cross section direction that results in the lowest factor of safety). Landslides that do not occur directly downslope and slopes where the direction of bedding dip is oblique to the slope require that consideration be given to the direction of failure. In general, the analyst can start the search for a critical failure direction by evaluating cross sections that extend directly downslope and directly down the dip of the failure surface or bedding plane and then expanding that search to include intermediate directions, if such appear to be more critical.

9.4 GROUNDWATER CONDITIONS

Engineers performing computer-aided slope stability analyses should determine how the specific program they are using accounts for pore-water pressure and be sure that they specify it correctly. For example, in the computer program XSTABL, when a phreatic surface is used to describe pore-water pressures and that phreatic surface is above the ground, a water surcharge is applied to the ground surface. However, when a piezometric surface is used in XSTABL and that surface is above the ground, no water surcharge is applied to the ground surface. Also, when specifying a phreatic surface in XSTABL, the program assumes that equipotential lines are

- If realistic soil compressibility data are available, FE/FD methods can give general information about deformations at working-stress levels.

- FE/FD methods illustrate progressive failure up to and including overall shear failure. By contouring shear strains in the zones, it is possible to highlight failure surfaces.

For non-linear analyses using complex constitutive models that attempt to reproduce volumetric changes accurately in undrained or partially drained conditions, the incremental application of gravity can produce different results than would be obtained if gravity is applied all at once. However, if a simplified elasto-plastic model is used in FE/FD analyses, the factor of safety appears unaffected (Griffiths and Lane, 1999). Therefore, if the primary goal of the FE/FD analysis is to obtain a factor of safety, a simplified Mohr-Coulomb elasto-plastic model can be used with an instantaneous gravity "turn-on" procedure (Griffiths and Lane, 1999). To determine the factor of safety (FS) from FE/FD analyses, the "shear strength reduction technique" can be used (Matsui and San, 1992). In that procedure, the FS of a soil slope is defined as the number by which the original shear strength parameters must be divided in order to bring the slope to the point of failure (as indicated by numerical non-convergence or excessive displacement). The "factored" shear strength parameters c'_f and ϕ'_f, are given by:

$$c'_f = c' / FS$$

$$\phi'_f = \arctan(\tan \phi' / FS)$$

The method would allow a different FS to be specified for the c' and tan ϕ' terms, but typically the same factor is applied to both terms. To find the slope's factor of safety, a systematic search is conducted to find the FS that initiates failure by solving the problem repeatedly using a sequence of user-specified FS values.

Modern FE/FD programs have enhanced graphical output capabilities that allow better understanding of the mechanisms of failure and simplify the output from reams of paper to useable graphs and plots of displacement. However, what remains is the concern that powerful tools such as the FE/FD method require considerable experience to properly evaluate the results.

The FE/FD method is a powerful tool which provides significant insight into the potential slope performance to the experienced user. A user should be thoroughly familiar with both the mathematical mode and the required input parameters before using this method.

slopes that are 2:1 in gradient or flatter should not, in the Committee's judgment, be required unless local experience indicates that slopes at that gradient commonly experience surficial instability.

terms of a median and standard deviation. Note that attenuation relations thus do not provide a specific value of the ground motion parameter. Therefore, even when a deterministic assessment of the causative earthquake is specified in terms of its magnitude and distance to the site, there is still a large range of potential ground motions that could occur as described by attenuation relations. Depending on the level of conservatism desired in deterministic analyses, typically either the median (50th percentile) or median-plus-one-standard-deviation (84th percentile) ground motion is used for design.

In the probabilistic approach, multiple potential earthquakes are considered. That is, all of the magnitudes and locations believed to be applicable to all of the presumed sources in an area are considered. Thus, the probabilistic approach does not consider just one scenario, but all of the presumed possible scenarios. Also considered are the rate of earthquake occurrence (how often each scenario earthquake occurs) and the probabilities of earthquake magnitudes, locations, and rupture dimensions. Moreover, the probabilistic approach considers all possible ground motions for each earthquake and their associated probabilities of occurring based on the ground motion attenuation relation.

The basic probabilistic approach yields a probabilistic description of how likely it is that different levels of ground motion will be exceeded at the site within a given time period, not merely how likely an earthquake is to occur. The inverse of the annual probability (i.e., the probability of exceedance for one year) is called the return period. Because probabilistic seismic hazard analyses sum the contribution of all possible earthquakes on all of the seismic sources presumed to impact a site, they do not result in a unique magnitude and distance that corresponds to the estimated acceleration value. Additional efforts are needed to extract the magnitude and distance most strongly contributing to the acceleration at a given hazard level. To estimate a magnitude and distance that can be paired with a given acceleration point (i.e., MHA and associated probability of exceedance), the hazard analysis for a given acceleration must be de-aggregated to develop the modal magnitude, $\overline{M}$, and modal distance, $\bar{r}$. Parameters $\overline{M}$ and $\bar{r}$ can be thought of as the magnitude and distance that contribute most strongly to the selected hazard level at the site. The process of de-aggregating the hazard to derive $\overline{M}$ and $\bar{r}$ is straightforward, but it must be understood that the de-aggregation is a function of hazard levels (i.e., different return periods). In addition, de-aggregation is sensitive to the ground motion parameter for which the hazard analyses are performed (i.e., different values of $\overline{M}$ and $\bar{r}$ could be obtained for MHA than for a long-period spectral acceleration).

There is a widespread misunderstanding of the relationship between deterministic and probabilistic analyses. Deterministic analyses are often (mistakenly) thought to provide "worst case" ground motions. That misunderstanding is a result of nebulous terminology that has been used in earthquake engineering. Terms such as "maximum credible earthquake" and "upper consistent with the UBC, ground motions should be obtained from a probabilistic seismic hazard analysis (PSHA).

Probabilistic seismic hazard analyses can be performed on a site-specific basis using available commercial computer codes. Alternatively, available CDMG maps can be used to estimate accelerations at different hazard levels. The CDMG maps can be useful provided the hazard level of interest is represented on the maps, there are not unusual soil conditions that could significantly affect ground motions (such as soft clay or peat), and the seismic source modeling used by CDMG remains appropriate (i.e., additional fault information compiled since publication of the CDMG maps has not rendered them obsolete). Estimation of peak accelerations using the state maps or site-specific analyses are discussed below.

10.2.1 State Maps

Ground motion maps are being created for each area affected by the California Seismic Hazards Mapping Act as a by-product of the delineation of Seismic Hazards Zones by the Department of Conservation. They form the basis of earthquake shaking opportunity in the regional assessment of liquefaction and seismically-induced landslides for zonation purposes. The maps are generated at a scale of about 1:150,000, using the MapInfo® street grid as the base. The maps are produced using a data-point spacing of about 5 kilometers (0.05 degrees), which is the spacing that was used to prepare the small-scale state ground-motion map used for the Building Code (Petersen et al., 1996; Frankel, 1996; Petersen et al., 1999).

Ground motions shown on the maps are expressed as maximum horizontal accelerations (MHA) having a 10-percent probability of being exceeded in a 50-year period (corresponding to a 475-year return period) in keeping with the UBC-level of hazard. Separate maps are prepared of expected MHA for three types of surficial geology (hard rock, soft rock, and alluvium), based on averaged ground motions from three different attenuation relations. When using those maps, it should be kept in mind that each assumes that the specific soil condition is present throughout the entire map area. Use of a MHA value from a particular soil-condition map at a given location is justified by the soil class determined from the site-investigation borings.

The set also includes a map of modal magnitude and distance pairs (i.e., $\overline{M}$ and $\bar{r}$) calculated at the same grid spacing as MHA. Those values represent the de-aggregated 475-year hazard level, and are available for the ground motion parameter of MHA for an alluvial site condition (the parameters are not sensitive to site condition, and hence the values on the maps can also be used for rock and soft rock site conditions). Because of the discrete nature of de-aggregated hazard, the user is cautioned not to interpolate modal parameters to the project site location when using

10.2.3 Site-Specific Deterministic Analyses

Deterministic analyses can be used to evaluate the seismic demand that would be placed on a site if a specific earthquake were to occur. If deterministic seismic hazard analyses are to be used to develop ground motion estimates, the following should be clearly documented in the project report: definition of the scenario earthquake, attenuation relationship used to evaluate ground motions for the scenario earthquake, and the percentile ground motion (e.g., 50th, 84th, etc.) that was selected. The engineer may wish to consult with the reviewing agency in developing these criteria for deterministic analyses. For non-critical structures, many engineers have used median ground motions from attenuation relations based on characteristic magnitudes associated with nearby faults; whereas for critical structures, 84th percentile ground motions have sometimes been used. In a region where an individual fault dominates the seismic hazard, the level of uncertainty to be used in prescriptive deterministic analyses can be estimated by performing probabilistic analyses and comparing the results with deterministic analyses at different uncertainty levels.

10.3 OTHER GROUND MOTION PARAMETERS

As noted at the beginning of this chapter, three ground motion parameters are needed for the evaluation of seismic slope stability – MHA, duration of strong shaking ($D_{5\text{-}95}$), and mean period (T_m). Of those, only MHA maps are currently available from CDMG. The focus of this section, therefore, is the estimation of $D_{5\text{-}95}$ and T_m for seismic slope displacement calculations.

The parameters $D_{5\text{-}95}$ and T_m are functions of magnitude (M), distance (r), and site condition (S=0 for rock, S=1 for soil). For a given M, r, and S, regression equations are available that provide a log-normal distribution of the $D_{5\text{-}95}$ and T_m parameters, not a single value. For use with the seismic slope displacement methodology discussed in Section 11.2, median values of $D_{5\text{-}95}$ and T_m can be used. Those values should be evaluated for the $\overline{M}$, $\bar{r}$ magnitude-distance pair (where $\overline{M}$ and $\bar{r}$ represent the 475-year hazard level for MHA). At their discretion, consultants may also wish to consider additional scenario earthquakes with larger magnitudes that might occur on major faults near the site. Once a magnitude-distance pair has been selected, median values of $D_{5\text{-}95}$ and T_m can be calculated as follows:

Duration (Abrahamson and Silva, 1996)

Median values of $D_{5\text{-}95}$ on rock can be estimated as follows. For r >10 km,

11 SEISMIC SLOPE STABILITY ANALYSIS

11.1 INTRODUCTION

11.1.1 Background

Recent practice for analysis of seismic slope performance has been to use a pseudo-static representation of seismic loading in a conventional limit-equilibrium analysis, or to perform a displacement analysis based on the analogy of a rigid block on an inclined plane (i.e., Newmark-type displacement analysis; Newmark, 1965).

There are two elements associated with a pseudo-static slope stability analysis procedure. First, a horizontal destabilizing seismic coefficient (k) must be specified, which represents the fraction of the weight of the slide mass that acts horizontally through the centroid of the mass. Second, a minimum acceptable factor of safety must be specified for the slope with the pseudo-static seismic force applied to it. In southern California, the most commonly used pseudo-static procedure is one adopted by Los Angeles County, and is modified from the recommendations of Seed (1979). The Seed procedure calls for $k = 0.15$ and FS ≥ 1.15, and was calibrated from Makdisi and Seed (1978) displacement analyses so as to produce slope deformations of one meter during magnitude 8.25 earthquakes. LA County has modified this procedure to have $k = 0.15$ and FS ≥ 1.10. Pseudo-static methods are recommended herein for the purpose of a screen analysis for slopes within hazard zones. However, the recommended procedures for screen analyses are modified from the Seed criterion to more properly account for the effects of seismicity on slope deformation hazard, and to recognize the relatively small deformation tolerance of typical hillside construction. These procedures are described in Section 11.2.

Newmark-type displacement analyses can be performed with two general methods. The first involves formal numerical integration of time histories of shaking within a slide mass according to the procedure described by Franklin and Chang (1977). The second method makes use of correlations between calculated Newmark displacements, selected ground motion parameters, and the ratio of seismic load resistance to peak demand (k_y/k_{max}, see definitions below). Several such correlations are available, including Makdisi and Seed (1978) and Bray and Rathje (1998).

T_m = mean period of input rock motion (sec)

T_s = fundamental period of equivalent 1-D slide mass at small strains (sec)

u = calculated slope displacement (in cm)

11.2 SCREENING ANALYSIS

11.2.1 Background

Seismic Hazard Zone maps published by the CDMG include Landslide Hazard Zones. Analyses of the type described in this chapter are required for sites located within those zones. The purpose of these analyses is to determine if the site has a significant seismic slope deformation potential. The mere fact that a site is within a Landslide Hazard Zone does not mean that there necessarily is a significant landslide potential at the site, only that a study should be performed to determine the potential.

The SP 117 Guidelines state that an investigation of the potential seismic hazards at a site can be performed in two steps: (1) a screening investigation and (2) a quantitative evaluation. The purpose of the screening investigation for sites within zones of required study is to filter out sites that have no potential or low potential for landslide development.

The screening criteria described in Sections 11.2.2 to 11.2.3 below may be applied to determine if further quantitative evaluation of landslide hazard potential is required. If the screening investigation clearly demonstrates the absence of seismically induced landslide hazards at a project site and the lead agency technical reviewer concurs, the screening investigation will satisfy the site investigation report requirement for seismic landslide hazards. If not, a more thorough quantitative evaluation will be required to assess the seismic landslide hazard, as described in Section 11.3.

11.2.2 Development of Screening Analysis Procedure

The screening analysis procedure recommended herein is based on a pseudo-static representation of seismic slope stability. The procedure is implemented by entering a horizontal seismic coefficient (k) into a conventional slope stability calculation. The seismic coefficient represents the fraction of the weight of the sliding mass that is applied as an equivalent horizontal force acting through the centroid of the mass. If the factor of safety is greater than one (FS > 1), the site passes the screen, and the site fails if FS < 1.

The seismic coefficient to be used in the analyses is taken as,

$$k_{eq} = f_{eq} \times (MHA_r / g) \qquad (11.1)$$

3. Factor k_{max} is related to $MHA_r \times NRF/g$, where NRF is a factor that accounts for the nonlinear response of the materials above the slide plane. Parameter $D_{5\text{-}95}$ is a function of magnitude and distance, as discussed in Section 10.3.

Based on the above, calculations were performed to evaluate for various combinations of MHA_r, magnitude, and distance, the f_{eq} values that cause the probability that seismic slope displacement would exceed 5 cm or 15 cm to be 50%. The Committee chose to use a 50% probability level because we believed probabilities departing significantly from 50% could significantly bias the effective return period from the standard 475-year hazard level. Additional details on this calculation are provided in Appendix A. The results of the calculations are shown in Figures 11.1(a) and 11.1(b) for the 5 cm and 15 cm threshold displacements, respectively.

The equation of the curves in Figure 11.1 is as follows:

$$f_{eq} = \frac{NRF}{3.477} \times \left[1.87 - \log_{10} \left(\frac{u}{(MHA_r / g) \times NRF \times D_{5\text{-}95}} \right) \right] \tag{11.2}$$

where u is in units of cm, $D_{5\text{-}95}$ = median duration (in seconds) from Abrahamson and Silva (1996) relationship (defined in Eq. 10.1) and NRF is defined by the relationship tabulated subsequently in Figure 11.2, which can be approximated by:

$$NRF \approx 0.6225 + 0.9196 \times Exp\left(\frac{-MHA_r / g}{0.4449} \right) \tag{11.3}$$

for $0.1 < MHA_r/g < 0.8$.

11.2.3 Screening Criteria

In summary, the following procedure is recommended for performing screening analyses for seismic slope stability:

1. Set up an analytical model for the slope as would normally be done for a static application, but with soil strengths that are appropriate for dynamic loading conditions. As noted in Chapter 7, this may require that different drainage conditions be considered than in the static case, and also requires consideration of rate effects and cyclic degradation on soil strength.

2. Use the procedures in Section 10.2 to estimate the maximum horizontal acceleration at the location of the site for a rock site condition (MHA_r). Parameter MHA_r should generally be evaluated using probabilistic seismic hazard analysis for a 475-year return period. Identify the mode magnitude ($\overline{M}$) and mode distance ($\bar{r}$) from de-aggregation of that hazard level.

3. Evaluate the site seismic coefficient using the procedures described in Section 11.2.2 with a value of threshold displacement that is considered acceptable by the local regulatory agency.

4. Perform a pseudo-static calculation of slope stability using the seismic coefficient from (3), and find the minimum factor of safety. Note that the critical failure surface will generally be shallower than the critical surface without a seismic coefficient.

5. Denote the factor of safety from (4) as FS. If FS > 1, the site passes the screen. However, for critical projects, consultants may want to perform additional checks for specific, large seismic sources in the local area, calculating M and r for each source deterministically. For each source considered, one would evaluate MHA_r and f_{eq} deterministically, and then check

MHA-*M*-*r* parameters can be translated into a more useful representation of demand for slope stability analysis.

The seismic loading for a potential sliding mass can be represented by the horizontal equivalent acceleration, HEA. HEA/*g* represents the ratio of the time-dependant horizontal inertia force applied to a slide mass during an earthquake to the weight of the mass. For a horizontal slide plane and horizontal ground surface, HEA can be calculated as:

$$HEA(t) = \left(\frac{\tau_h(t)}{\sigma_v} \right) g \qquad (11.4)$$

where *t* indicates that there is time variation, τ_h is the horizontal shear stress at the depth of the sliding surface calculated by a one-dimensional seismic site response analysis program (e.g., SHAKE91, Idriss and Sun, 1992; D-MOD, Matasovic, 1993), and σ_v is the total vertical stress at the depth of the sliding surface. For more complex geometries (i.e., not one-dimensional), a rigorous analysis of HEA requires the use of two-dimensional finite element analyses (e.g., QUAD4M; Hudson et al., 1994). Rathje and Bray (1999a) have found that 1-D analyses generally provide a conservative estimate of HEA(*t*) for deep sliding surfaces and a slightly unconservative estimate for shallow surfaces near slope crests. MHEA is the maximum horizontal equivalent acceleration over the duration of earthquake shaking. For slope displacement analyses, seismic demand is typically represented by HEA time histories or MHEA coupled with duration $D_{5\text{-}95}$.

The seismic demand in a slide mass can be relatively rigorously evaluated from two dimensional finite element dynamic response analyses using a program such as QUAD4M (Hudson et al., 1994). Those analyses enable the evaluation of HEA time histories that are customized to the specific geometry and soil condition at the subject site. The analyses should be performed using sets of at least 5-10 time histories as input. Those time history sets should be appropriate for the magnitude and site-source distance that control the site hazard. Fewer time histories (3-4) can be used if they are scaled to match the constant hazard spectrum for the site (established from a site-specific probabilistic seismic hazard analysis) across the period range of interest (e.g., Richardson et al., 1995; Kavazanjian et al., 1997). Further discussion on time histories for slope displacement analyses is provided in Section 11.3.3.

A second procedure represents the amplitude of seismic demand with MHEA. The procedure was developed by Bray et al. (1998) from statistical analysis of many wave propagation results in equivalent one-dimensional slide masses. The procedure normalizes MHEA in the slide mass by the product of MHA_r and a nonlinear response factor (NRF). Parameter NRF accounts for nonlinear ground response effects as vertically propagating shear waves propagate upwards

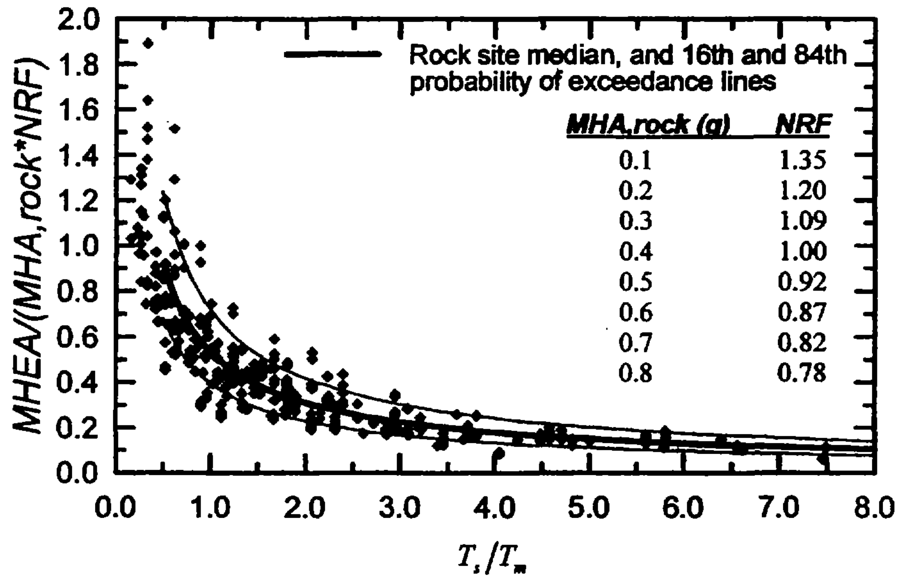


Figure 11.2. Normalized MHEA for Deep-Seated Slide Surface Vs. Normalized Fundamental Period of Slide Mass (after Bray et al., 1998).

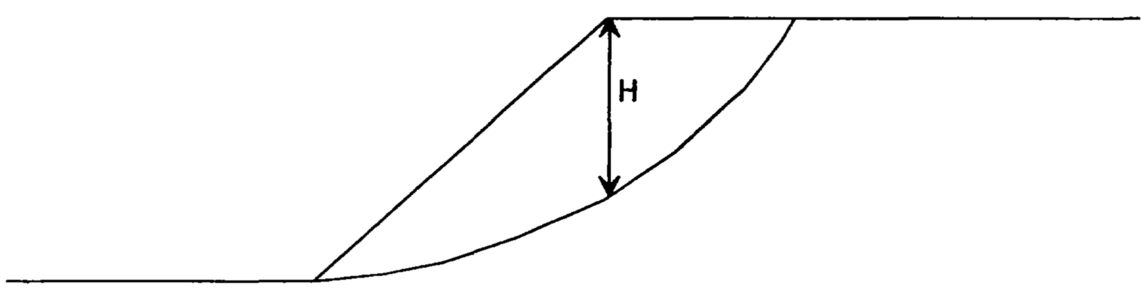


Figure 11.3. Definition of Height of Slide Mass for Use in Equation 11.5

11.3.3 Estimation of Seismic Slope Displacements

Two possible quantifications of demand for slope stability calculations were described in Section 11.3.2:

- Use of a simplifying assumption to evaluate MHEA = $k_{max} \cdot g$.
- Use of dynamic analysis to define time histories of horizontal equivalent acceleration, HEA(t).

The second method for estimating slope displacement utilizes the recommendations of Makdisi and Seed (1978) for relating k_y/k_{max} to displacement u. Parameter k_{max} for application in the Makdisi and Seed procedure is not evaluated using the methods described in Section 11.2.2. Rather, the MHA at the crest of a triangular embankment section is evaluated, and k_{max} is estimated using Figure 11.5. The Committee is not aware of simplified procedures for evaluating the crest MHA for typical fill slope geometries, which are not triangular in cross-section. Such an evaluation would need to consider ground response effects through the slide mass and topographic effects. A consultant using the Makdisi and Seed approach should reach an agreement with the cognizant public official regarding an appropriate procedure for evaluating this crest acceleration, as well as a procedure for evaluating k_{max} from crest acceleration for non-triangular slope geometries.

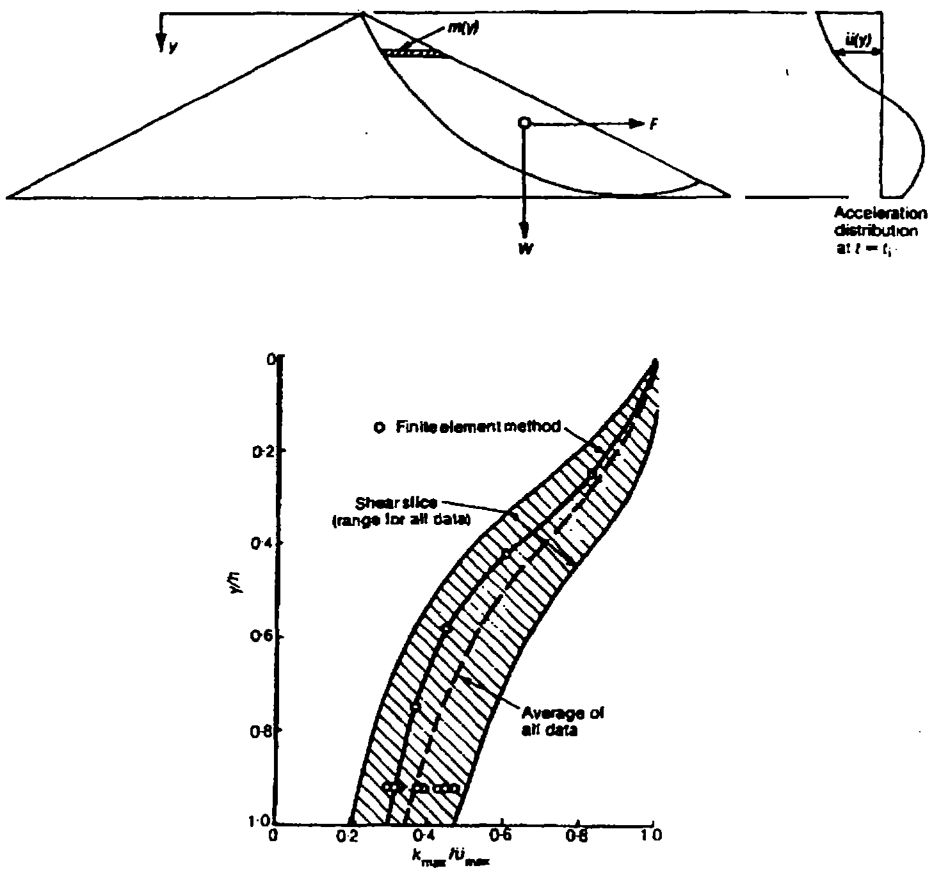


Fig. 11.5. Variation of k_{max} with Depth in Triangular-Shaped Embankment Section (Makdisi and Seed, 1978). Parameter $\ddot{u}_{max}$ Denotes Peak Acceleration at Embankment Crest.

As noted previously in Section 11.3.2, Newmark displacement analyses should generally be performed using HEA time histories, because such motions account for the effects of ground motion amplification and incoherence through the slide mass. However, there are a limited number of cases where Newmark analyses can be performed using as-recorded accelerograms as estimates of HEA time histories. As recommended by Rathje and Bray (1999b), this practice is acceptable for very short period slide masses having $T_s/T_m < 0.2$.

Finally, it should be noted that the identification of the most critical slip surface for seismic slope displacement analysis depends not only on the slope/material properties (as is the case under static conditions), but also on the variation of shaking in the slope. What is desired is the k_y/k_{max} combination that yields the largest slope displacement. In many cases, this will be the critical surface identified from the calculations described in Section 11.3.1. Shallower surfaces should be checked, however, because while they will have higher k_y values, they may also have larger k_{max} values, which could lead to larger displacements. The Committee considers the use of shallower surfaces to be unnecessary if $\text{MHEA}/(\text{MHA}_r \times \text{NRF}) = 1.0$. However, if $\text{MHEA}/(\text{MHA}_r \times \text{NRF})$ is less than 1.0 (see Figure 11.2), at a minimum, one additional surface should be considered and it is the deepest surface that produces $\text{MHEA}/(\text{MHA}_r \times \text{NRF}) = 1.0$ (note that this will be shallower than the surface having the lowest k_y).

11.3.4 Tolerable Newmark Displacements

The final step in the analysis is to decide if the calculated displacement is acceptable. Ideally, allowable displacements for analyses would be established from a database in which observed slope displacements from earthquakes are correlated to measures of damage in structures associated with the slope displacements. Unfortunately, however, such data do not exist in sufficient quantity to be useful, and hence there is no rational basis for selecting allowable displacements. Accordingly, allowable displacement levels are established from engineering judgment. The judgment of the majority of the Committee is that if the critical slip surface from slope stability analyses daylights within a structure that is likely to be occupied by people during an earthquake, the median displacements (u) should be maintained at less than 5 cm. A minority of the Committee feels that those displacements through occupied structures should be maintained at less than 15 cm. Neither of these values (5 or 15 cm) is necessarily the "correct" value, because they are judgment-based. Individual agencies may wish to select their own allowable displacement values based on their experience and judgment. No matter which allowable displacement values are selected, the procedures described in the preceding sections can be readily applied with those threshold displacements.

The scope of this Committee's activities, and the Seismic Hazards Mapping Act, does not extend beyond inhabited structures. However, owners, engineers, or cognizant public officials may, at

12 SLOPE STABILITY HAZARD MITIGATION

Slopes that possess factors of safety less than required by the governing agency, or with unacceptably large seismic slope displacements, require avoidance or mitigation to improve their stability. Even if a slope is found from analyses to be stable, it might require protection in order to avoid degradation of shear strengths from weathering, to remain stable under future increased loading conditions, to prevent toe erosion, or to remain stable under future, potentially higher groundwater conditions than assumed in the analyses. Protection for adjacent pad areas may also be required to minimize hazard from erosion and falling debris.

The most common methods of mitigation are (1) hazard avoidance, (2) grading to improve slope stability, (3) reinforcement of the slope or improvement of the soil within the slope, and (4) reinforcement of the structure built on the slope to tolerate the anticipated displacement. Avoidance involves placing a proposed improvement a sufficient distance from an unstable slope. Grading methods commonly employed to improve slope stability include partial or complete replacement of unstable soil. Slopes can be strengthened with soil reinforcement, retaining walls, deep foundations, geosynthetics, and/or soil nails/tiebacks can be used alone or in conjunction with grading to improve slope stability. Soil can be improved with cement or lime stabilization. Structures built on slopes also can be sufficiently reinforced to reduce damage to a tolerable amount. In addition, structures can be effectively isolated from ground deformations through the use of piles or compaction grouting.

The mitigation measures chosen for a given slope must be analyzed recognizing that different mitigation measures require analyses for different modes of failure. Some methods (for example, slope reinforcement) require consideration of strain compatibility and soil/structure and/or soil material interaction issues. The following sections describe both stabilization and mitigation measures, and the potential modes of failure that should be analyzed.

Creation of a temporary backcut is usually required when performing partial or total removal and replacement. The backcut must be analyzed and designed to have a sufficient static factor of safety during construction, typically 1.25, to allow the safe construction of the permanent slope

12.2.3 Stability Fills

A stability fill is used where a slope has an adequate factor of safety for gross stability, but an insufficient factor of safety for surficial stability or where the materials exposed at the slope surface are prone to erosion, sloughing, rock falls, or other surficial conditions that require remediation. Stability fills are relatively narrow, typically about 10 to 15 feet wide. Soil placed in the stability fill should be compacted to at least 90 percent of the maximum density as determined by ASTM D1557, unless a different degree of compaction is recommended by a Geotechnical Engineer and approved by the governing agency. Water content also should be controlled during compaction, because fills compacted to water contents wetter than the line of optimums have been shown to perform significantly better than fills compacted to lower water contents in both static and seismic conditions (Lawton et al., 1989; Whang, 2001). A higher percent relative compaction may be required for steeper slopes and coarse-grained soil types. That can be facilitated by overbuilding the slopes and trimming them back to the compacted core (which is preferable to rolling the surface of the slope).

Stability fills should be keyed into firm underlying soil or competent bedrock. The key should be at least as wide as the stability fill and should extend at least 3 feet below the toe of the slope. Both the gross and surficial stability of the stability fill should meet the minimum stability requirements set by the governing agency. The gross or deep-seated stability should be analyzed along failure surfaces extending through the toe of the slope and beneath the keyway. Combinations of circular and non-circular failure surfaces should be used as applicable.

12.2.4 Buttress Fills

A buttress fill provides the features of a stability fill, but is used where a slope does not have a sufficient factor of safety for gross or deep-seated stability and additional resistive forces are required. For example, buttress fills can be used to support upslope landslides or slopes in sedimentary rock where the bedding is adversely dipping out of the slope.

The base of a buttress fill is typically wide, usually ranging from about one third to almost the full height of the slope being buttressed. The actual width of the buttress must be determined by slope stability analysis. Soil placed in the buttress fill should be compacted to a minimum of 90 percent of the maximum density as determined by ASTM D1557, unless a different degree of compaction is recommended by a Geotechnical Engineer or required by the governing agency. Water content also should be controlled, as discussed in Section 12.2.3. Buttress fills should be

Chimney drains can be provided every 25 to 50 linear feet at the interface of the stabilization fill and natural ground to enhance the backdrain system performances. The purpose of a chimney drain is to collect subsurface water from multiple bedding planes. The use of chimney drains is particularly important for buttress fills that will support bedded rock with considerably different permeability between layers. Conventional near-horizontal subdrains often will not collect water from the permeable layers because they do not intersect or cross the permeable beds. The chimney drains should be continuous between lateral backdrains and should be a minimum of 2 feet in width. Chimney drains may be created by stacking gravel-filled burlap (not woven plastic) bags, placement of a continuous gravel column surrounded by non-woven filter fabric, or placement of a drainage composite. Drain locations and outlet pipes should be surveyed in the field at the time of installation.

12.3 ENGINEERED STABILIZATION DEVICES AND SOIL IMPROVEMENT

A grading solution to a slope stability problem is not always feasible due to physical constraints such as property-line location, location of existing structures, the presence of steep slopes, and/or the presence of very low-strength soil. In such cases, it may be feasible to mechanically stabilize the slide mass or to improve the soil with admixture stabilization. The resulting slope should be analyzed to meet the same requirements as other slopes.

Mechanical stabilization of slopes can be accomplished using retaining walls, deep foundations (i.e., piles or drilled shafts), soil reinforcement with geosynthetics, tieback anchors, and soil nails. Common admixture stabilization measures include cement and lime treatment as well as Geofibers™.

12.3.1 Deep Foundations

The factor of safety of a slope can be increased by installing soldier piles/drilled shafts through the unstable soil into competent underlying materials. The piles/drilled shafts are sized and spaced so as to provide the required additional resisting force to achieve adequate slope stability. The piles/drilled shafts typically provide resistance through the bending capacity of the shaft anchored by passive resistance in stable earth materials underlying the slide mass.

The load applied to the deep foundation from material above the potential failure surface is commonly represented using a uniform or equivalent fluid pressure (triangular) distribution. Resistance to failure is provided by passive earth pressure within the "stable earth materials." In this context, stable earth materials are defined as those materials located beneath the potential failure surface having a static FS $\geq$ 1.5 and along which the anticipated seismic displacement is less than 5 cm or 15 cm (with the effects of the deep foundations and any other stabilization devices such as tieback anchors excluded in the analysis). In general, no resistance should be deflections of the deep foundations are of concern, deflections can be calculated based on soil properties evaluated using unfactored soil strengths. Soldier piles/drilled shafts used to stabilize the slope and provide support for a structure should be tied in two lateral directions such that the potential for lateral separation is minimized.

12.3.2 Tieback Anchors

The loads on the soldier piles/drilled shafts are, in some cases, higher than these elements can support in cantilever action alone. Tieback anchors can be incorporated in those cases to provide additional resistance. Tieback anchors also can be used without soldier piles/drilled shafts by anchoring them against a wall or reinforced face element. Tieback anchors consist of steel rods or cables that are installed in a drilled, angled holes. The rods/cables are grouted in place within the reaction zone and extend through a frictionless sleeve in the unstable mass. The anchors are post-tensioned after the grout reaches its design strength. Anchors are often tested to a load that is higher than the design load. The anchors must be long enough to extend into stable earth materials as defined in Section 12.3.1.

Temporary anchors generally do not need to be protected from corrosion. Permanent anchors should be protected from corrosion for the design life of the project. A reference for the design of ground anchors is Sabatini et al. (1999).

12.3.3 Soil Nails

Soil nailing involves earth reinforcement by placing and grouting reinforcing rods in holes drilled in the ground. The reinforcing rods are not pre-stressed or post-tensioned. Soil nailing should not be used in relatively fines-free gravel and sandy soil. A reference for the design of soil nails is Bryne et al. (1996). Soil nailing for permanent slope stabilization has been widely used by CalTrans and FHWA in Public Works projects. The application of this technique for general use is currently being studied by a special committee in southern California.

12.3.4 Retaining Structures

A retaining wall can be constructed through an unstable slope to provide additional resistance and raise the factor of safety for material behind the wall to an acceptable level. Retaining structures should be founded in stable earth materials as defined in Section 12.3.1. The retaining structure should be evaluated for possible sliding, overturning, and bearing failures using standard techniques. Failure surfaces that extend below the wall foundation and above the top of the wall also should be analyzed. Analysis of walls that support bedded rock dipping toward the wall is facilitated by use of a computer program that also allows the use of anisotropic strength parameters. Consideration must be given to whether material in front of the wall that is assumed

The effectiveness of dewatering drains or wells needs to be checked periodically by measuring the water levels in the slope. Drains and wells, whether pumped or static, require periodic maintenance to assure that the casing does not become clogged by fines or precipitates and that the pump is functioning. The effectiveness of subsurface drainage control features is dependent on proper maintenance of the drains and/or wells. Where proper maintenance of the wells/drains cannot be guaranteed for the time period during which the stability of the slope is to be maintained, a dewatering system should not be relied upon to achieve the required factor of safety.

"Passive" dewatering with subdrains was discussed previously in section 12.2.6.

12.5 CONTAINMENT

Loose materials, such as colluvium, slopewash, slide debris, and broken rock, on the slope that could pose a hazard can be collected by a containment structure capable of holding the volume of material that is expected to fail and reach the containment device over a given period of time. The containment structure type, size, and configuration will depend on the anticipated volume to be retained and the configuration of the site. Debris basins, graded berms, graded ditches, debris walls, and slough walls can be used. In some cases, debris fences may be permitted, although those structures often fail upon high-velocity impact.

The expected volume of debris should be estimated by the geologist and engineer. Debris walls and slough walls should be designed for a lateral equivalent pressure of at least 125 pounds per cubic foot where impact loading is anticipated and at least 90 pounds per cubic foot elsewhere unless otherwise allowed by the regulatory agency and/or justified by the consultant. The height of the catchment devices may be governed by the expected debris volume of the expected bounce height of a rolling rock. The CRSP program (Jones, et al., 2000) can be used to estimate rolling rock trajectories.

Access should be provided to debris containment devices for maintenance. The type of access required is dependent on the anticipated volume of debris requiring removal. Wheelbarrow access will be sufficient in some cases, whereas heavy equipment access may be required in other areas.

12.6 DEFLECTION

Walls or berms that are constructed at an angle to the expected path of a debris flow can be used to deflect and transport debris around a structure. The channel gradient behind those walls or berms must be sufficient to cause the debris to flow rather than collect. Required channel gradients may range from 10 to 40 percent depending on the expected viscosity of the debris and

13 CONCLUDING REMARKS

This document has presented a broad overview of landslide hazard analysis, evaluation, and mitigation techniques. The Implementation Committee acknowledges that the state of the art in slope stability evaluation continues to evolve and advance and that new methodologies in geotechnical engineering, soil/shear strength testing, slope-stability analysis, and mitigation will develop.

Many of the issues germane to this topic, such as strength evaluation and the treatment of uncertainties, were the subjects of extended debate by the Committee. Typically at issue was the pervasive use in current practice of antiquated technologies that provide misleading, or at best highly uncertain, outcomes. All too often, the Committee was compelled to adopt language encouraging (or at least allowing) the use of such technologies when more robust (but invariably more expensive) alternatives exist. One important example of this is the use of direct shear strength testing of samples from Modified California samplers. Another is the continued use of a static FS=1.5 regardless of the level of subsurface characterization and project importance. Technologies currently exist, and continue ~~to be developed,~~ that allow geotechnical engineering practice to move beyond gross conservatism and almost ~~purely judgment-based~~ design. What is needed is clear recognition by consultants, regulators, and owners of the economic and societal benefits of proper geotechnical work. If the provisions in this document are adopted in practice, it will represent a small step in the right direction, but all parties involved must remain diligent in trying to advance the all too often tradition-bound profession we share.

The implementation of SP 117 represents an important step in furthering seismic safety in the State of California. Proper analysis of both the static and seismic stability of slopes is critical to the safety and well being of Californians as development continues to expand into hillside areas. It is the hope of the Implementation Committee that this document will make a contribution toward that goal and provide useful information and guidance to owners, developers, engineers, and regulators in the understanding and solution of the slope stability and landslide hazards that exist in California and in other tectonically active regions.

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ABSTRACT

Site-specific seismic slope stability analyses are required in California by the 1990 California Seismic Hazards Mapping Act for sites located within mapped hazard zones and scheduled for development with more than four single-family dwellings. A screen analysis is performed to distinguish sites for which only small ground deformations are likely from sites for which larger, more damaging landslide movements could occur. No additional analyses are required for sites that pass the screen, whereas relatively detailed analyses are required for sites that fail the screen. We present a screen analysis procedure that is based on a calibrated pseudo-static representation of seismic slope stability. The novel feature of the present screen procedure is that it accounts not only for the effects of ground motion amplitude on slope displacement, but also accounts for duration effects indirectly via the site seismicity. This formulation enables a more site-specific screen analysis than previous formulations that made *a priori* assumptions of seismicity/duration.

reduces the pseudo-static factor of safety (FS) for a given slope to unity, and is referred to as the yield acceleration, k_y. The second is the peak value of spatially averaged horizontal acceleration (normalized by g) across the slide mass, and is denoted k_{max}.

Perhaps the most widely used screen analysis procedure is that developed by Seed (1979) for application to earth dams. The procedure calls for k = 0.1 or 0.15 to be applied for M = 6.5 and 8.25 earthquakes, respectively. The screen is passed if the factor of safety, FS, exceeds 1.15. A slightly modified version of that procedure, in which k = 0.15 and FS ≥ 1.1 regardless of local seismicity, was adopted in 1978 by Los Angeles County for application to hillside residential construction. Seed (1979) recommended that his procedure only be applied for cases where the earth materials do not undergo significant strength loss upon cyclic loading (i.e., strength loss < 15%) and where several feet of crest displacement was deemed "acceptable performance," as is the case for many earth dams (e.g., 0.9 m displacement for M = 8.25 and crest acceleration = 0.75g).

An important feature of the Seed (1979) procedure is its calibration to a particular slope performance level, which is represented by the displacement of a rigid block on an inclined plane (i.e., a "Newmark-type" displacement analysis, Newmark, 1965). Seed (1979) calibrated his pseudo-static approach using Newmark displacements calculated with simplified methods (e.g., Makdisi and Seed, 1978). The Makdisi and Seed simplified procedure, in turn, is based on a limited number of calculations that were used to relate Newmark displacement to earthquake magnitude and k_y/k_{max} (e.g., five calculations for M = 6.5, two for M = 7.5, and two for M = 8.25). Seed's (1979) recommendations are an important milestone, as they represent the first calibration of a pseudo-static method to a particular level of slope performance as indexed by displacement. This concept underlies other widely used screen analysis procedures that have been developed to date, and is retained as well in the present work.

Since the Seed (1979) work, additional screen analysis procedures have been developed for application to earth dams and solid waste landfills. A procedure for earth dams was developed by Hynes-Griffin and Franklin (1984) based on (1) calculations of shaking within embankment sections using a linear elastic shear beam model by Sarma (1979) and (2) calculations of Newmark displacement from time histories using the analysis approach of Franklin and Chang (1977). Those calculations resulted in statistical relationships between the amplification of shaking within embankments (i.e., ratio of $k_{max} \times$ g to maximum horizontal acceleration of base rock, MHA_r) and the depth of the sliding surface, as well as between Newmark displacement and k_y/k_{max}. Hynes-Griffin and Franklin (1984) developed their pseudo-static procedure using approximately a 95th percentile value of amplification for deep sliding-surfaces along with the upper-bound value of k_y/k_{max} that produces 1.0 m of displacement. In the resulting procedure, k is taken as $0.5 \times MHA_r$, and the screen is passed if FS ≥ 1.0. The procedure is intended for use with 80% of the shear strength in non-degrading materials. The method is not recommended for

The screen analysis procedure developed herein is intended principally for application to hillside residential and commercial developments. For construction of this type, small ground deformations can cause collateral loss that is considered unacceptable by owners, insurers, and regulatory agencies. Accordingly, the limiting displacements used in existing screen procedures for earth dams and landfills are considered to be too large for application to hillside construction. Another problem with the existing procedures is the level of conservatism employed in their development. For example, the existing methods apply for specific ranges of earthquake magnitude (which are high for the Seed and Bray et al. methods), and may not pass otherwise safe sites for which the design magnitude is smaller than that used in the development of the screen. Moreover, the conservative interpretation of amplification and displacement distributions used in the development of existing schemes likely makes the level of risk associated with the slope performance differ significantly from that associated with the ground motions. In other words, if the ground motion is evaluated with probabilistic hazard analysis for a given return period, and the slope displacement conditioned on that ground motion is extreme (i.e., a rare realization), the resulting slope design is based on displacements having a much longer return period than the design-basis ground motion.

Given those shortcomings, the Committee has developed a new screen procedure tailored to the needs of hillside residential and commercial construction (in terms of displacement) and which accounts for site-specific seismicity. The screen procedure was also developed so as to control the level of conservatism in order to maintain a reasonable return period on the expected slope performance. The remainder of this appendix describes the development of the procedure.

DEVELOPMENT OF SCREEN ANALYSIS PROCEDURE

Introduction

The purpose of screen investigations for sites within zones of required study is to filter out sites that have no potential or low potential for earthquake-induced landslide development. No additional seismic stability analysis is required for a site that passes the screen, whereas further quantitative evaluation of landslide hazard potential (and possibly mitigation) is required for sites that fail the screen.

Like other screen procedures described in the previous section, ours is based on a pseudo-static representation of seismic slope stability. The procedure is implemented by entering a de-stabilizing horizontal seismic coefficient (k) into a conventional slope stability analysis. The seismic coefficient represents the fraction of the weight of the sliding mass that is applied as an equivalent horizontal force acting through the centroid of the mass. If the factor of safety is greater than one (FS > 1), the site passes the screen, and the site fails if FS < 1.

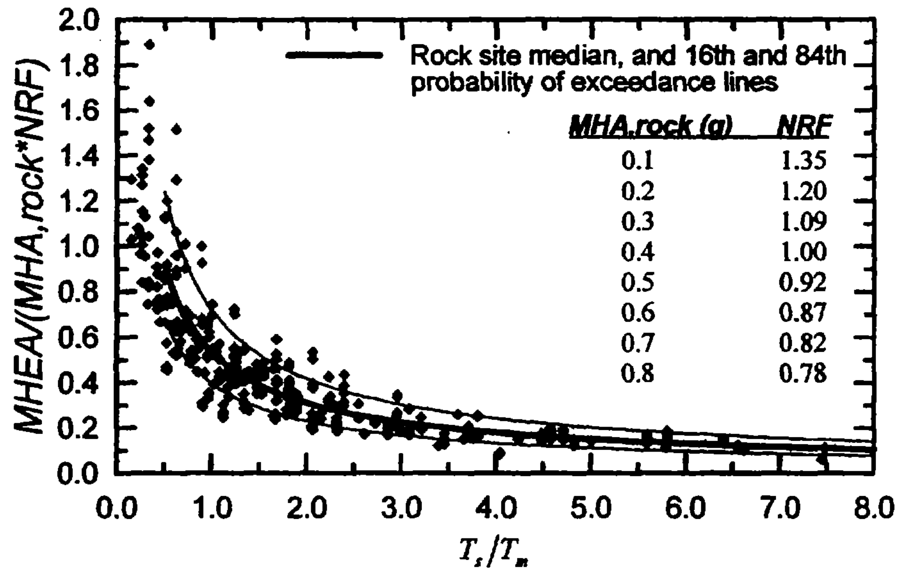

Fig. 1. Normalized MHEA for Deep-Seated Slide Surface vs. Normalized Fundamental Period of Slide Mass (after Bray et al., 1998).

The magnitude and distance that control the peak acceleration hazard in much of urban southern California are magnitude 6.5 – 7.0 earthquakes at distances generally less than 10 km (Petersen et al., 1996). Parameter T_m has a median value of about 0.5 s for these magnitude and distance ranges (Rathje et al., 1998). Parameter T_s is calculated as

$$T_s = \frac{4H}{V_s} \qquad (3)$$

where H = thickness of slide mass and V_s = average shear wave velocity of slide mass. If V_s is taken as 300 m/s (consistent with soft bedrock or compacted fill materials), the slide mass thickness would have to exceed about 20 m for $T_s/T_m > 0.5$. It was therefore the Committee's judgment that MHEA/($MHA_r \times$ NRF) = 1.0 would be a reasonable assumption for sites having critical slip surfaces of moderate to shallow depth (< ~20 m), and would be conservative for deeper-seated slip surfaces (depth > ~20 m). Because parameter NRF is a function of MHA_r (as shown in Figure 1) the assumption of MHEA/($MHA_r \times$ NRF) = 1.0 makes MHEA solely a function of MHA_r. Accordingly, Eq. 2 can be re-written as Eq. 1 provided the effect of NRF is incorporated into factor f_{eq}, which is done in the next section.

Formulation of Seismicity Factor f_{eq}

For a given MHA_r, large magnitude earthquakes will tend to cause poorer slope performance than smaller magnitude earthquakes. One important reason for this is that large magnitude earthquakes have longer durations of shaking. Previous pseudo-static procedures for seismic slope stability have specified a single value for f_{eq}, and thus have made implicit, and usually very

A relationship between magnitude, distance, MHA_r, and f_{eq} was established using the Bray and Rathje relationship with the following assumptions and observations:

1. Factor f_{eq}^* (Eq. 2) was taken as equivalent to k_y/k_{max}. The equivalency of k_y/k_{max} and f_{eq}^* can be understood by recognizing that k_y/k_{max} simply represents the factor by which the actual ground shaking intensity (k_{max}) needs to be reduced to render a seismic coefficient associated with FS = 1 (i.e., $k_y = k_y/k_{max} \times k_{max}$). Referring to Eq. 2, because our screen procedure is intended for use with FS = 1, f_{eq}^* represents the factor by which MHEA/g needs to be reduced to yield a seismic coefficient associated with FS = 1 (i.e., k_y). Accordingly, if k_y is substituted for k in Eq. 2 (appropriate for FS = 1) and k_{max} is substituted for MHEA/g, it can be readily seen that $f_{eq}^* = k_y/k_{max}$.

2. Parameter MHEA is inconvenient for use in a screen procedure because its relationship to MHA_r is affected by vertical ground motion incoherence effects and nonlinear ground response effects. As described in the previous section, to simplify the analysis we neglect the vertical incoherence effects, which is equivalent to assuming MHEA/($MHA_r \times$ NRF) = 1.0. From Eq. 1 and 2, we see that $f_{eq} = f_{eq}^* \times$ MHEA/MHA_r, which reduces to $f_{eq}^* \times$ NRF with the above assumption. Since $f_{eq}^* = k_y/k_{max}$, we calculate parameter $f_{eq} = k_y/k_{max} \times$ NRF.

3. Two threshold levels of Newmark displacement were selected by the Committee, u=5 and 15 cm. It should be noted that the Newmark displacement parameter is merely an index of slope performance. The 5 cm threshold value likely distinguishes conditions for which very little displacement is likely from conditions for which moderate or higher displacements are likely. The 15 cm value likely distinguishes conditions in which small to moderate displacement are likely from conditions where large displacements are likely. It should be noted that those threshold displacement values are smaller than values used in the development of existing screen procedures for dams and landfills. The Committee's use of the small displacement value is driven by a concern on the part of owners, insurers, and regulatory agencies to minimize collateral loss from slope deformations in future earthquakes.

4. Factor k_{max} is taken as $MHA_r \times$ NRF/g. Parameter D5-95 is a function of magnitude and distance, and can be estimated from available attenuation relationships.

Based on the above, calculations were performed to evaluate as a function of f_{eq} the probability that seismic slope displacement u > 5 cm conditional on MHA_r, magnitude, and distance. This probability is calculated as:

$$P(u > 5cm \mid MHA_r, M, r, f_{eq}) = \int_{D_{5-95}} f(D_{5-95} \mid m, r) P(u > 5cm \mid D_{5-95}(M,r), MHA_r, f_{eq}) d(D_{5-95}) \qquad (5)$$

The distribution of median f_{eq} values with *M*, *r*, and MHA_r are shown in Figure 4(a) for u = 5 cm and in Figure 4(b) for u = 15 cm. The values in Figures 4 were derived using the Abrahamson and Silva (1996) attenuation model for duration at rock sites. Near-fault effects on ground motion parameters were neglected in the development of Figures 4; such effects would tend to increase the amplitude of long-period components of the ground motion but decrease the duration, and hence the net effect on seismic slope displacements would likely be small. Focal mechanism does not affect these calculations because the Abrahamson and Silva attenuation model for duration does not contain a focal mechanism term.

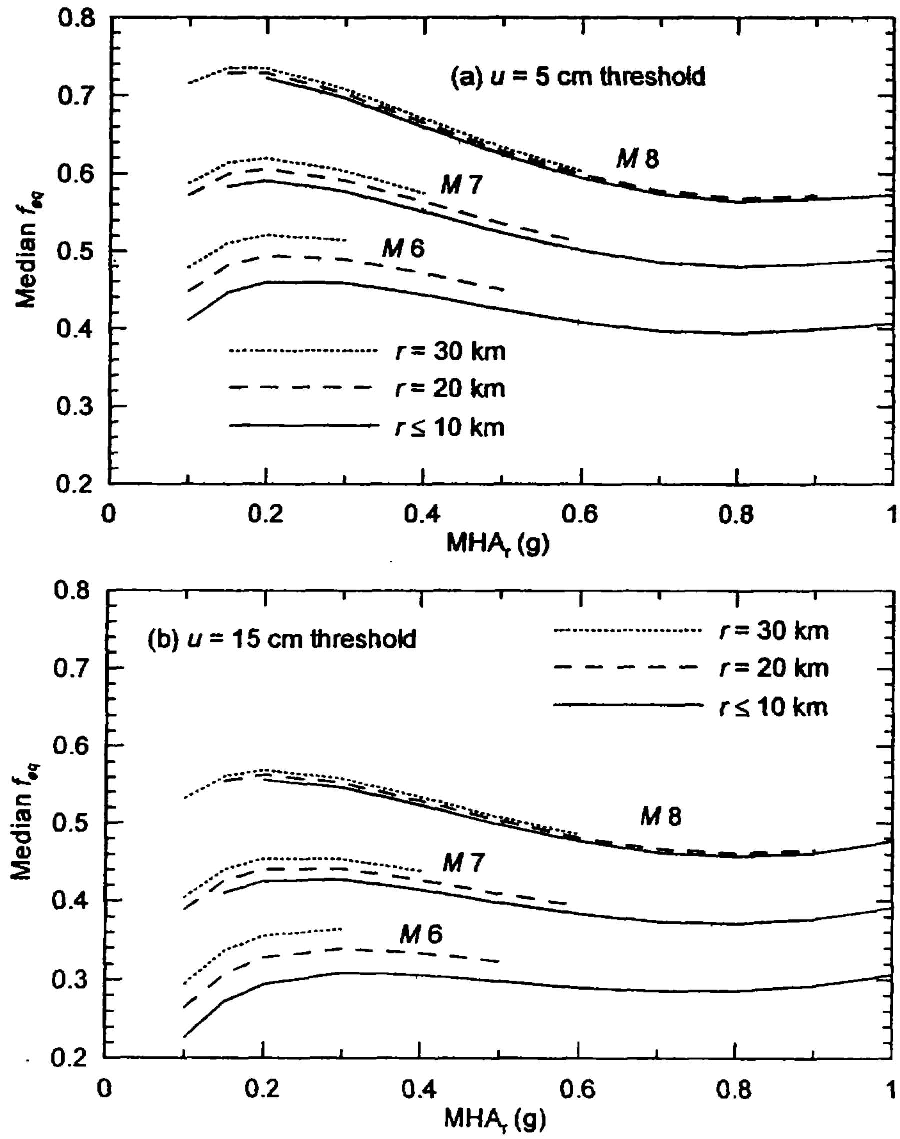

Fig. 4. Required Values of f_{eq} as Function of MHA_r and Seismological Condition for Acceptable Slope Performance

seismic hazard analysis (PSHA). The relative contributions of earthquake events at different magnitudes and distances to this MHA_r hazard should then be evaluated through a de-aggregation analysis, and the mode magnitude ($\overline{M}$) and mode distance ($\bar{r}$) identified for use in the screen. That combination of MHA_r, $\overline{M}$, and $\bar{r}$ represents the parameters that should be used to evaluate k. The Committee considered the use of supplemental deterministic seismic hazard analyses for sites located near large-magnitude, high slip-rate faults (such as the San Andreas fault system). However, it was found for many checked locations that k values computed deterministically were less than k values evaluated from PSHA. The PSHA results used in those checks are from published State-wide maps (Petersen et al., 1996). In our checks, the deterministic k values were evaluated using the characteristic earthquake event (as compiled by Petersen et al., 1996) on the largest fault segment nearest the site, and the 84th percentile MHA_r value associated with that characteristic event. The Committee recognizes that more severe deterministic scenario events could be conceived, but those would likely be sufficiently rare as to have a return period that significantly exceeds the 475-year target.

Limitations

As with other screen analysis procedures, the present procedure should not be used for slopes comprised of geologic materials that could be subject to significant strain softening, such as liquefiable soil. The procedure is not applicable to slopes constructed over soft clay soil, because as noted previously the Bray et al. (1998) relationship for MHEA (Figure 1) does not apply for that site condition. The procedure also should not be applied to situations for which 5 cm (or 15 cm) displacement is an inappropriate displacement threshold. Finally, it should be noted that this screen analysis procedure, and any analysis of seismic slope stability based on Newmark sliding block models, only provides an index of slope performance that is related to the accumulation of permanent shear deformations within the ground. Volumetric ground deformations associated with post-liquefaction pore-pressure dissipation or seismic compression of unsaturated ground are not considered in Newmark-type models and need to be evaluated separately.

Examples

Seismic coefficients (k) for three example sites in southern California are evaluated to illustrate application of the screen procedure defined by Eqs. 1 and 6. Locations of the sites are shown in Figure 5. The site denoted "Los Angeles" in Figure 5 is on the north flank of the Santa Monica Mountains, and is not immediately adjacent to any major active fault systems. The site denoted "Glendale" is near the base of the San Gabriel Mountains, and is close to the Sierra Madre fault system. The site at the intersection of Highway 138 and Interstate Highway 5 is adjacent to the San Andreas fault.

It should also be noted that the $\overline{M}$ values indicated in Table 1 are consistent with the characteristic earthquake magnitudes for faults near the respective sites (as tabulated in Petersen et al., 1996). The similarity of those magnitudes is the principal reason that the Committee does not consider it necessary to perform supplemental deterministic analyses of scenario events (which would have a magnitude similar to the characteristic earthquake magnitude).

Post-Screen Analysis

For sites that fail the screen analysis, more detailed slope displacement calculations should be performed. Several alternative analysis procedures are recommended by the Committee. Those include simplified analysis of Newmark displacement using the procedures formulated by Makdisi and Seed (1978) or Bray and Rathje (1998), or formal Newmark analysis of sliding block displacements using appropriate integration techniques with applicable earthquake time histories. Those procedures are well documented in the literature, and are summarized in Chapter 11 of the attached report.

CONCLUSIONS

In this appendix, we have presented a screen analysis procedure for seismic slope stability that takes into account local variations in seismicity, as represented by the magnitude (M) and distance (r) that most significantly contribute to the ground motion hazard at a site. The screen procedure is based on a statistical relationship previously developed by Bray and Rathje (1998) between seismic slope displacement (u), peak amplitude of shaking in the slide mass (k_{max}), significant duration of shaking ($D_{5\text{-}95}$), and the ratio of slope resistance to peak demand (k_y/k_{max}). The screen is formulated to separate sites expected to undergo small to negligible slope deformation from sites where larger and more damaging slope movements are likely. Application of the screen is straightforward. Pseudo-static seismic coefficient k is calculated using Eq. 1, with the parameter f_{eq} in Eq. 1 evaluated using Figure 4 based on the site seismicity and the tolerable slope displacement.

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APPENDIX C

GROUND WATER MODELING AND LEVEL PROJECTIONS

HANSEN ALLEN & LUCE INC
ENGINEERS

CLIENT: Wasatch Regional
PROJECT: Wasatch Regional Landfill Permit
FEATURE: Modeling Model - G.W. Simulations
PROJECT NO: 113.30.100

SHEET 1 OF 17
COMPUTED: [illegible]
CHECKED: [illegible]
DATE: 9/7/2004

PROBLEM: CREATE A G.W. MODEL OF THE WASATCH REGIONAL LANDFILL TO DETERMINE MAXIMUM POTENTIAL G.W. ELEVATIONS UNDER THE PROPOSED FACILITY.

DATA:

- Groundwater Observations from borings at facility by Kleinfelder ... 2003 (SEE SHEET 17)
- Tech Pub. No 42 (Stephens, 1974)
- Precip. data from the Desert Research Institute's Western Regional Climate Center (http://www.wrcc.dri.edu)
- USGS 7½ minute topographic quadrangles
  - Craner [illegible]
  - Badger Island NW
  - Delle
  - Poverty Point

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  Evapotranspiration
  Recharge Estimates
  Drain
  Hydraulic Conductivity
MODEL CALIBRATION
MODEL RESULTS
  Maximum Estimated [illegible]
  Drain Trench along Full Length of Facility
  Drain Trench for Initial Phase

ATTACHED - MAP SHOWING MODEL COORDINATE SYSTEM

STUDY AREA:

The study area includes the proposed Landfill site located in Sections 33 & 34, Township 2 North, Range 8 West, SLB&M and " " 3 & 4, " 1 ", " 8 ", west of the Great Salt Lake in Tooele County.
The facility is located west of the railroad and east of the foot of the Lakeside Mountains.

MODEL DISCRETIZATION

[illegible] x = 4,527,221

In order to define the MODFLOW MODEL, a coordinate system was established running parallel with section lines, with the northeast corner of Section 28, T.2 N., R.7 W., SLB&M being coincident with point x = 5,000′ y = 23,000′ in the coordinate system. The model grid consists of square cells with 500 ft per side. There are 46 rows and 74 columns. The west edge of Column 1 coincides with the coordinate x = 0′ and the north edge of row 1 coincides with y = 23,000′. The coordinate system is shown on the attached (folded) map. North & South boundaries of the model were chosen at least 1 mile north & South of the facility to avoid boundary effects on the target area to be modeled. Due to limited data, the area is modeled as 1 single layer.

BOUNDARY CONDITIONS

The western boundary is modeled as a Specified flux boundary using positive flowrate (injection) wells to simulate recharge from the bedrock and mountain streams of the Lakeside Mountains. The eastern boundary is modeled as a specified head boundary simulating the constant elevation of the Great Salt Lake. Under Existing conditions with the lake level @ elev ≈ 4195′, the lake boundary is at x ≈ 37,000′. Under projected future high lake level conditions (elev ≈ [illegible]) the lake boundary is at about x = 16,000′.
The northern & southern model boundaries are modeled as no flow boundaries simulating the west to east flow of groundwater, as indicated in Tech. Pub. No 42 ([illegible])
[illegible]

MODEL INPUT

Layer Elevations

The top elevation of the model was determined using the topographic contours of the USGS 7½-minute quads. The bottom elevation ranges from 100 ft below the top elevations on the west to 400 ft below the top elevations on the east. The thickness of the unconsolidated valley fill is certainly greater than 400 feet on the east but layer [illegible] were modeled using hydraulic conductivity. Therefore, since the bottom elevation is well below the lake level, and hydraulic conductivity is used instead of transmissivity, the bottom [illegible] should not have a significant impact on model results.

Great Salt Lake Elevations

Current and historical maximum GSL elevations were obtained from the USGS website:

ut.water.usgs.gov/gslelevgraphs/elevations.html

Current elevation = 4195 feet

Max elevation (1984-85) = 4212 ft.

(SEE SHEET 4)

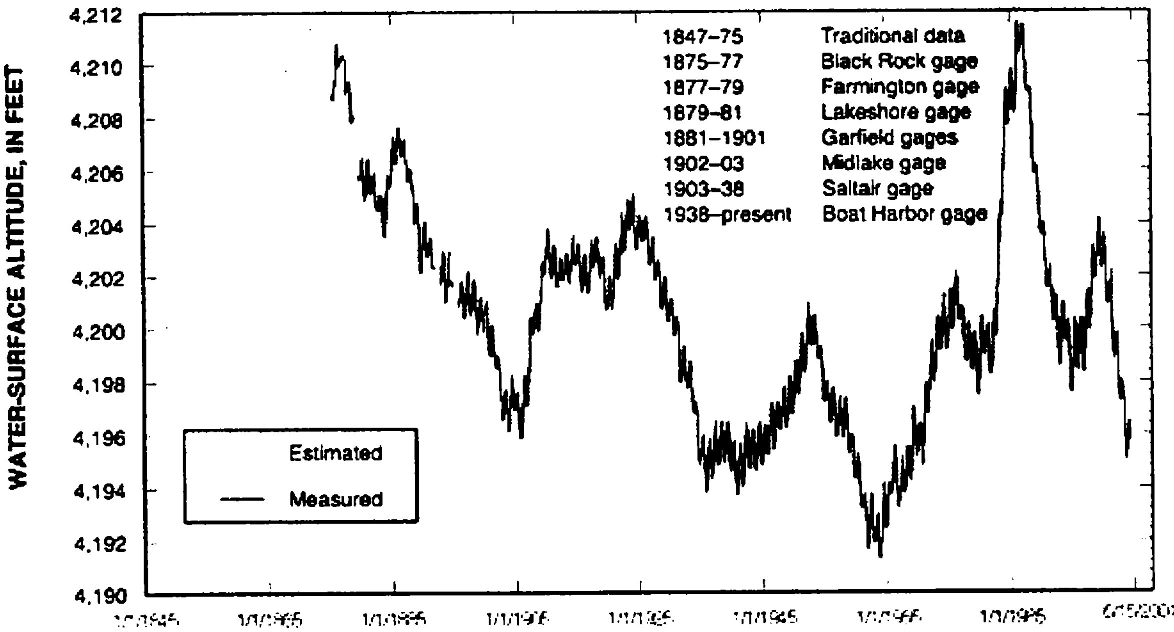
Evapotranspiration

Evapotranspiration was assumed to occur east of the facility. The ET elevation (elevation @ max ET rate) was assumed to be the ground surface. The extinction depth was assumed to be 5 feet (no ET below this).
The max ET rate was obtained from the average annual evapotranspiration for cell closure conditions presented in the HELP Model results summary from the September 2004 HAL calculations titled "HELP Model Input Summary."

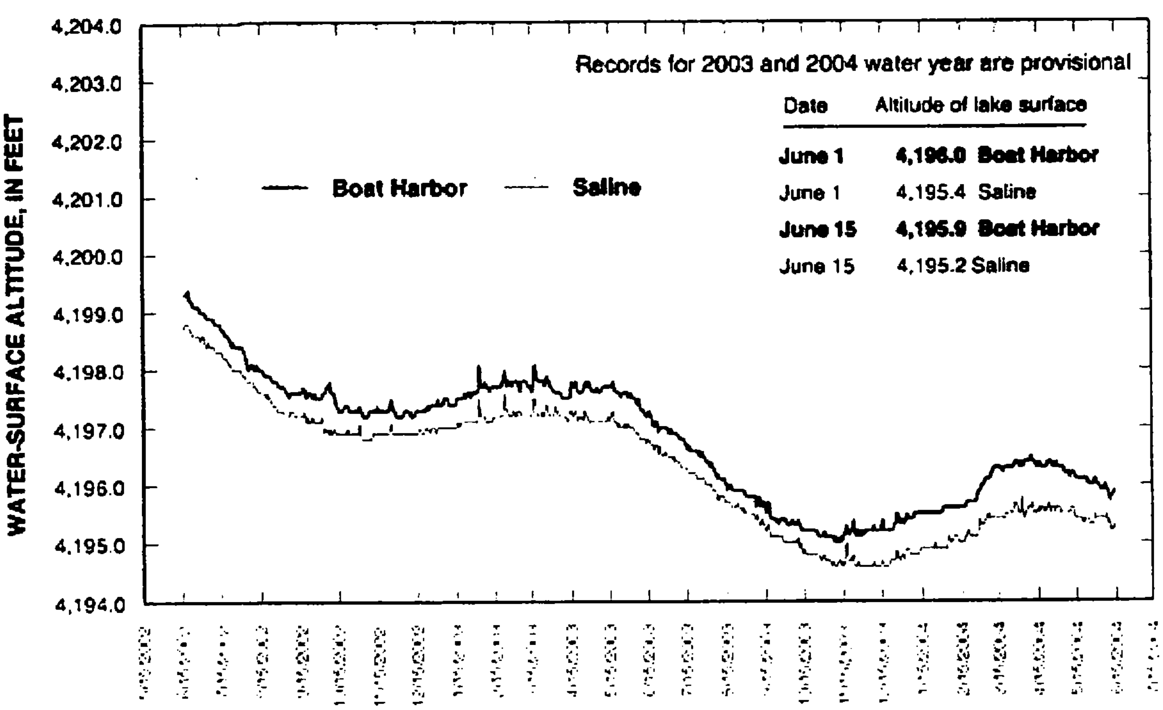
Max ET rate ≈ 12 in/yr [illegible]
= 0.0033 [illegible]

4/17

USGS
science for a changing world



Fluctuation in water-surface altitude of Gilbert Bay (south part), Great Salt Lake, 1847 to present



Fluctuation in water-surface altitude of both parts of Great Salt Lake during last 2 years

HANSEN ALLEN & LUCE INC ENGINEERS			
CLIENT	Wasatch Regional	SHEET	5 OF 17
PROJECT	Wasatch Regional Landfill Permit	COMPUTED	[illegible]
FEATURE	Modflow Model - GW Simulations	CHECKED	GMA
PROJECT NO	113.30.100	DATE	8/25/04

Recharge Estimates:

RECHARGE ZONES:

Divide Recharge into 3 zones & assume all recharge is from Lakeside Mtns West of study area.

North Recharge Area: (SHEET 6) — Carter Canyon Drainage, AREA = 94,240,000 ft²

Central Recharge Area: (SHEET 7) — Drainages South of Carter Canyon to Dead Cow Point. AREA = 109,600,000 ft²

South Recharge Area: (SHEET 8) — South of Dead Cow Point, AREA = 49,280,000 ft²

PRECIPITATION:

Based on Tech Pub No. 42 (Stephens, 1974), the average percent of precipitation contributing to groundwater recharge for the periphery of the Northern Great Salt Lake Desert, which includes the Lakeside Mtns, is 3%. Because the Lakeside Mountains aren't specifically addressed in T.P.-42, this analysis conservatively assumed 5% of precipitation contributes to recharge. The 4 closest precipitation stations to the study area from the Western Regional Climate Center website (www.wrcc.dri.edu) by the Desert Research Institute are:

Period of Record	Sta. Name	Lat	Lon	Elev
09/1990 – 12/2003	Utah Test Range	41°03'	112°56'	4440'
05/1986 – 12/2003	Knolls 10 NE	40°44'	113°12'	4240'
05/1967 – 10/1990	Callister Ranch	40°41'	112°40'	4260'
01/1956 – 12/2003	Grantsville	40°36'	112°27'	4290'

Use Grantsville, Knolls 10 NE, & Utah Test Range to obtain average precipitation from 1999 to 2003 (Record for all station)

	1999	2000	2001	2002	2003	Avg
Utah Test Range	X	X	5.09	6.95	9.54	7.1 in
Grantsville	X	11.85	X	7.08	6.92	8.6 in
Knolls 10 NE	X	3.78	X	X	5.2	4.4 in

Ave = 6.7
= 0.56 ft/yr

Use Callister Ranch & Grantsville to obtain average precipitation under [illegible] expected conditions from 1980 – 1983

	1980	1981	1982	1983	Avg
Callister Ranch	15.73	13.07	16.88	16.50	15.5 in
Grantsville	16.67	13.05	18.5	20.78	16.2 in

Ave = 15.9 in
= 1.325 ft/yr

X = [illegible]

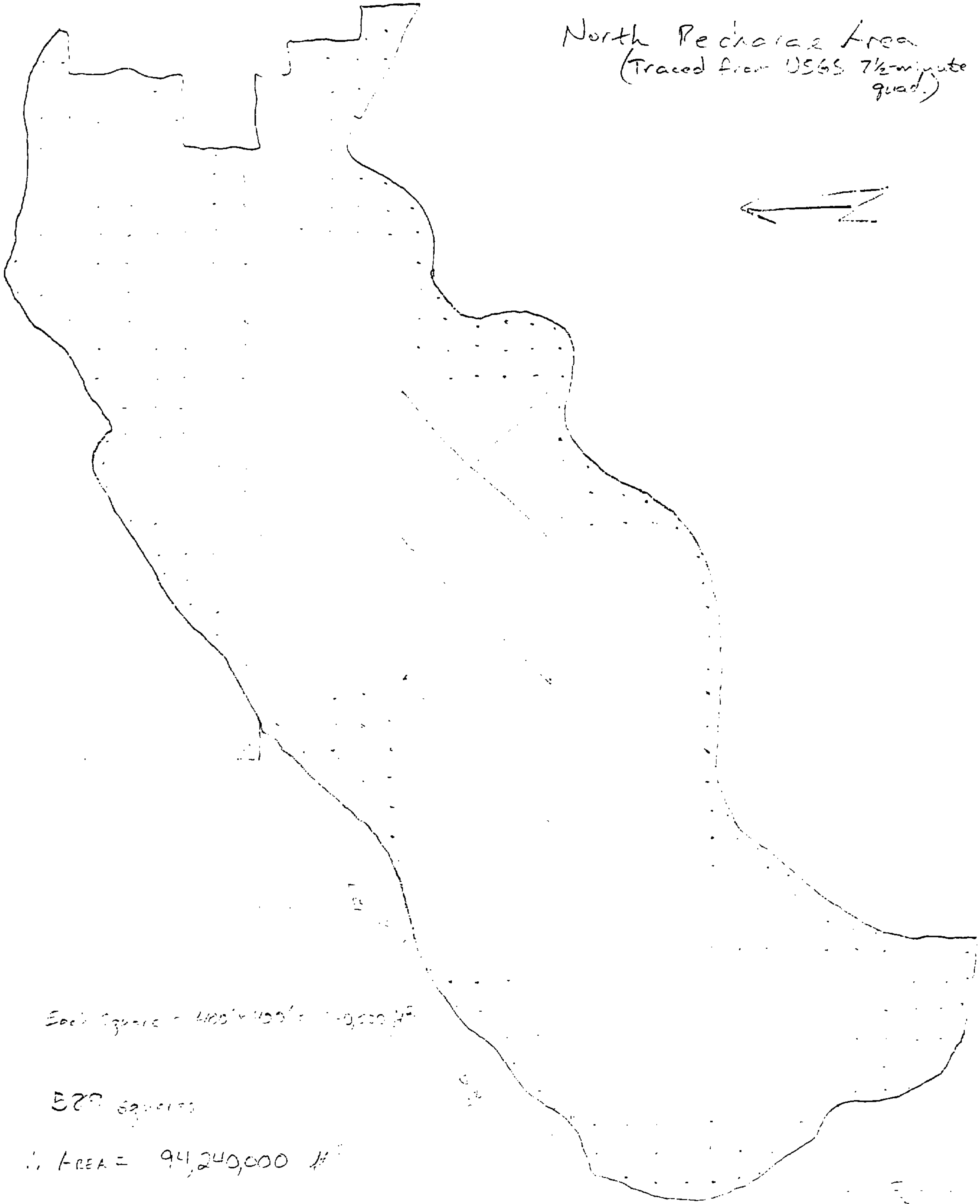
HANSEN ALLEN & LUCE INC ENGINEERS

CLIENT Wasatch Regional
PROJECT Wasatch Regional Landfill Permit
FEATURE Modflow Model - GW Simulations
PROJECT NO 113.30.100

SHEET 6 OF 17
COMPUTED [illegible]
CHECKED [illegible]
DATE 8/25/94

1" = 2000'

North Recharge Area
(Traced from USGS 7½-minute quad.)



Each Square = 400' x 400' = 160,000 ft²

589 squares

∴ Area = 94,240,000 ft²

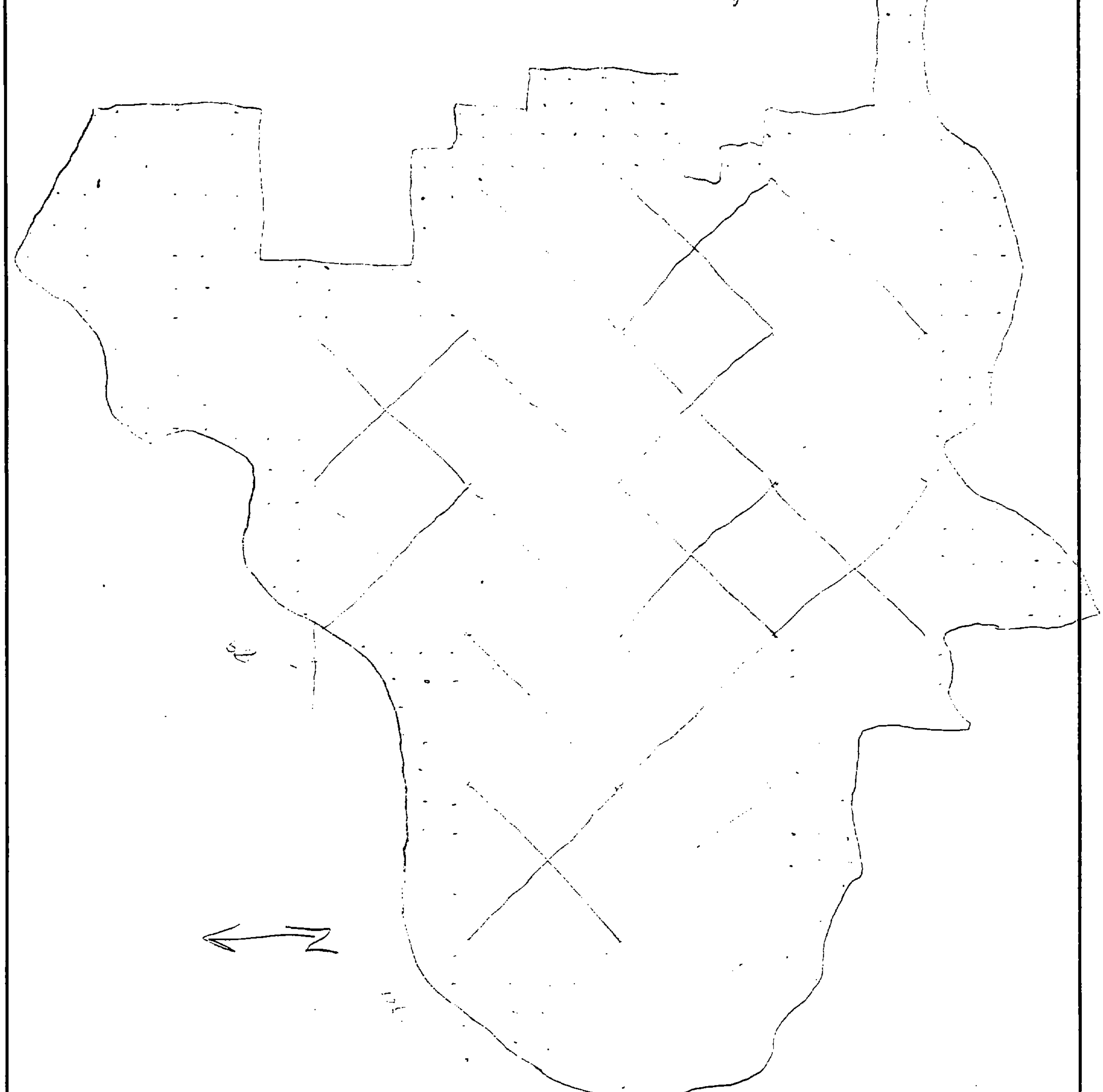
HANSEN ALLEN & LUCE inc
ENGINEERS

CLIENT Wasatch Regional
PROJECT Wasatch Regional Landfill Permit
FEATURE Nitrate Model - GW Simulations
PROJECT NO 113.30.100

SHEET 7 OF 17
COMPUTED
CHECKED
DATE 8/23/04

1" = 2000'

Central Recharge Area
(traced from USGS 7½-min. quad)



Each Square = 400' × 400' = 160,000 ft²

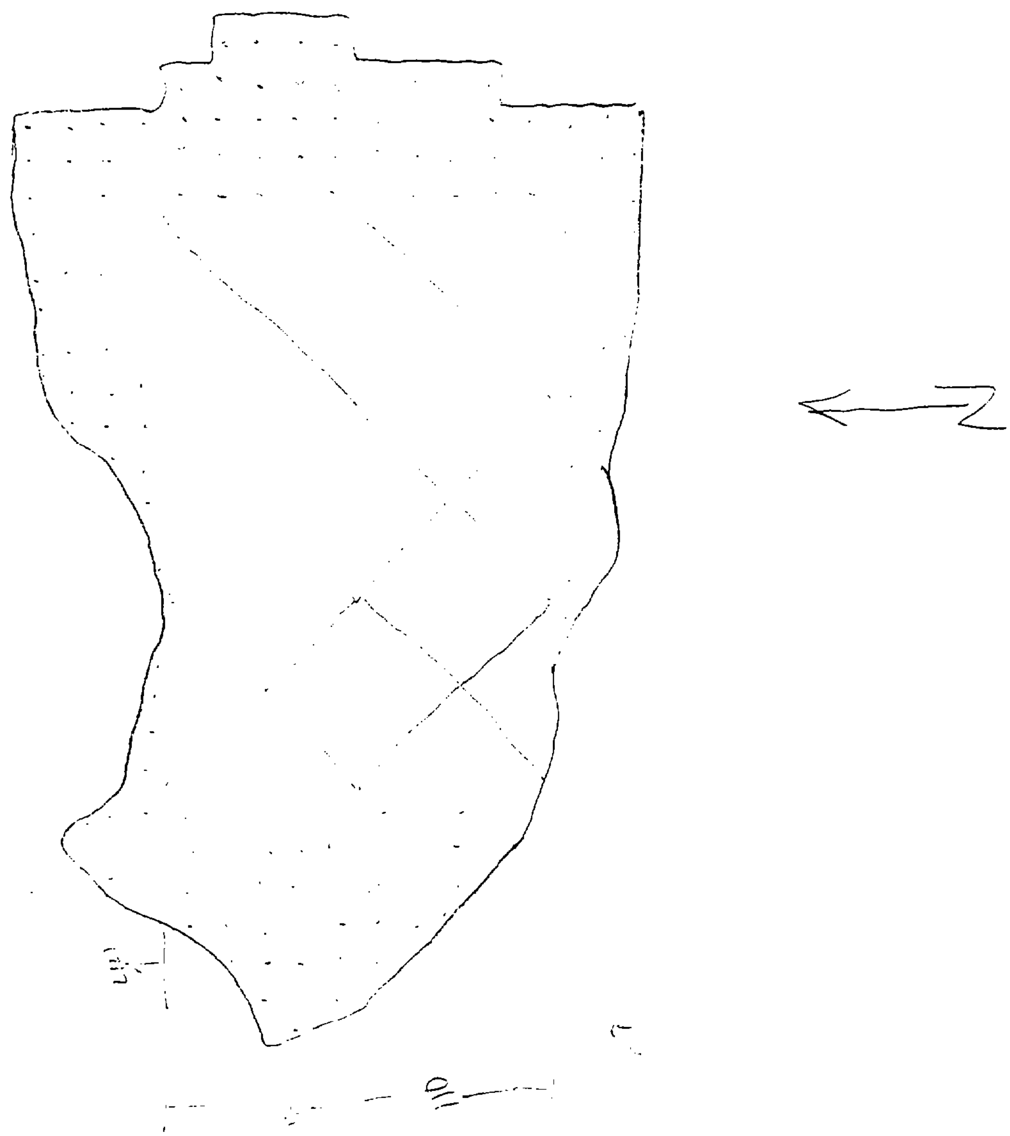
685 Squares

AREA = 109,600,000 ft²

CLIENT	Wasatch Regional	SHEET	8 OF 17
PROJECT	Wasatch Regional Landfill Permit	COMPUTED	[illegible]
FEATURE	Modflow Model - GW Simulations	CHECKED	[illegible]
PROJECT NO	113.30.100	DATE	8/25/04

1" = 2000'

South Recharge Area
(Traced from USGS 7 1/2-min quad)



Each Square = 400' x 400' = 160,000 ft²

[illegible] squares

AREA = 49,280,000 ft²

HANSEN ALLEN & LUCE inc ENGINEERS	CLIENT: Wasatch Regional PROJECT: Wasatch Regional Landfill Permit FEATURE: Modflow Model - GW Simulations PROJECT NO: 113.30.100	SHEET 9 OF 17 COMPUTED: [illegible] CHECKED: [illegible] DATE: 8/25/04 to 9/07/04

Total Precipitation by Recharge Area

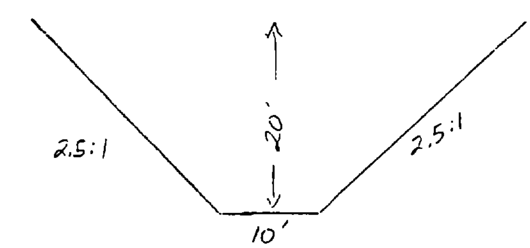
		North	Central	South
1999-2003	Total Volume (ft³/day)	144,587	168,153	75,608
	Volume Recharge (ft³/day)	7,229	8,408	3,780
	# cells	12	21	13
	Average Volume/cell (ft³/day)	602	400	291
1980-1983	Tot. Vol. (ft³/day)	342,104	397,863	178,893
	Vol. Recharge (ft³/day)	17,105	19,893	8,945
	# cells	12	21	13
	Average Volume/cell (ft³/day)	1425	947	688

Concentrate more of the recharge at mouths of canyons. Distribution by fraction of the average volume/cell (shown above) is shown on SHEET 10. For example, in the north recharge area the cells at the mouth of Carter Canyon have 2 times the average volume/cell and the cells furthest from the mouth of Carter Canyon have half the average volume/cell so the overall volume for the recharge area is unchanged.

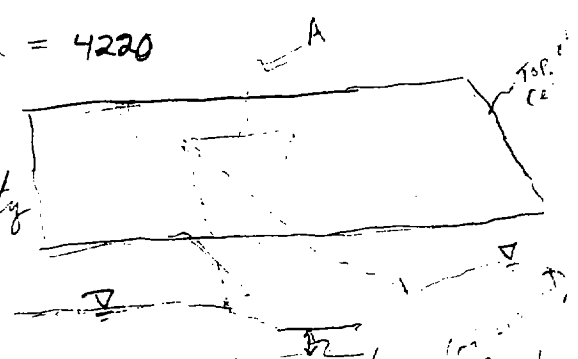
DRAIN (for future construction to control groundwater)

Model an open trench @ low end of facility as a drain

Cross-section



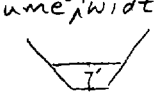
Lowest elevation of Drain = 4220



Conductance = $C = \frac{kA}{L}$

k = hydraulic conductivity
A = plan area of drain
L = flow length through bed of drain

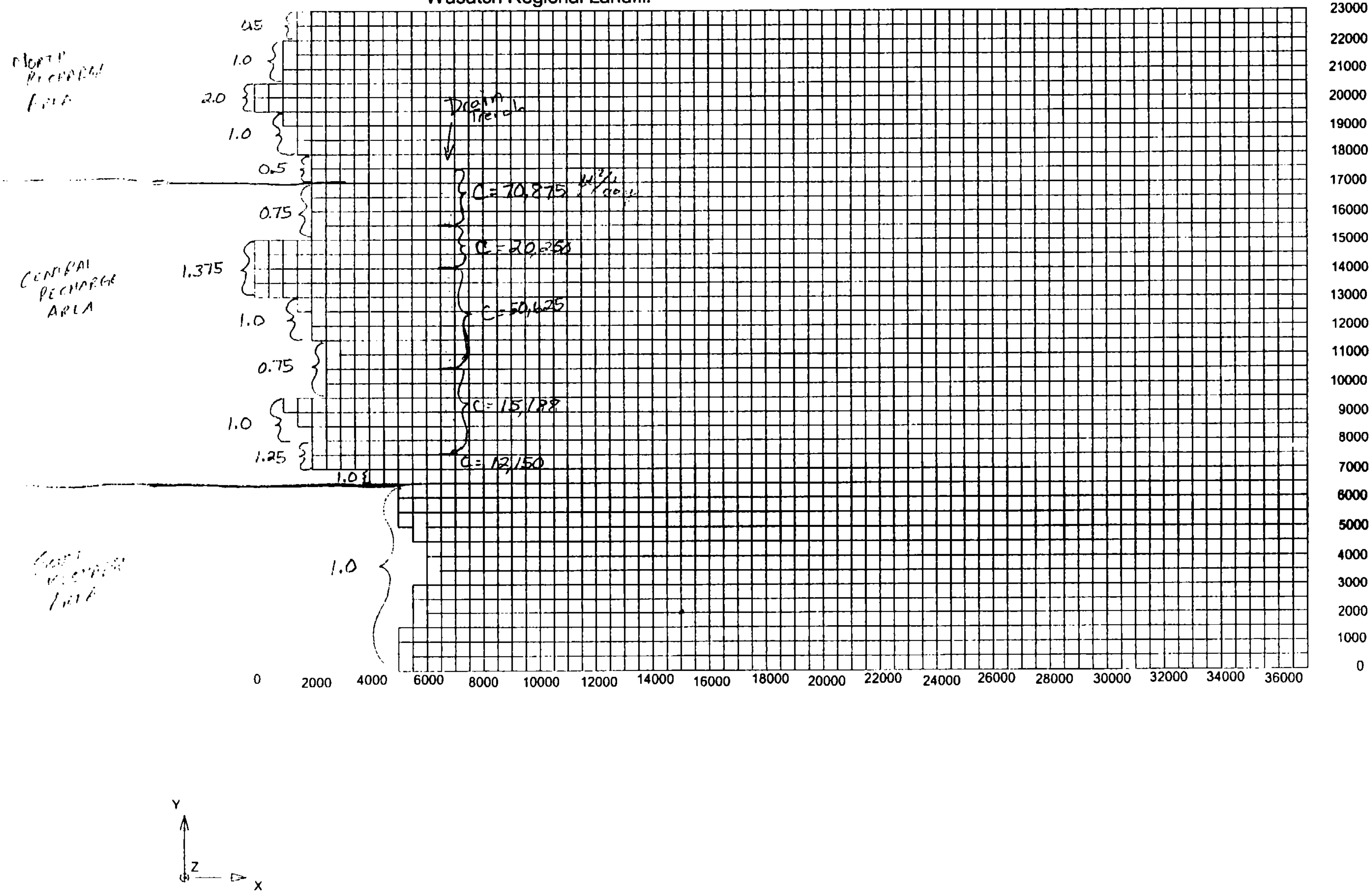
k - Due to soil disturbance from construction, use 90% of model K for drain bed.
A - assume top width of cross-section above @ depth of 7' & length of 1 model cell (500')



Top width = 7×2×2.5 + 10 = 45'

L - assume a drain bed thickness of 2 feet (maximum impacted by construction)

Wasatch Regional Landfill -



10/17

DRAIN (continued)

Model in the column of cells between 6,500′ and 7,000′ of the study area grid (column 14 or J:14) which is just east of the proposed landfill from row 12 (I:12) to row 32 (I:32) (or 7000′ to 17,500′ of the grid)

$$C = \frac{k(500\,ft)(45\,ft)}{2\,ft} = (11{,}250\,ft)(k) \Rightarrow \text{resulting conductance per cell}$$

(SEE SHEET 10)

Row(s)	MODEL K (ft/day)	k (ft/day)	C (ft²/day)
12-15	7	6.3	70,875
16-18	2	1.8	20,250
19-25	5	4.5	50,625
26-31	1.5	1.35	15,188
32	1.2	1.08	12,150

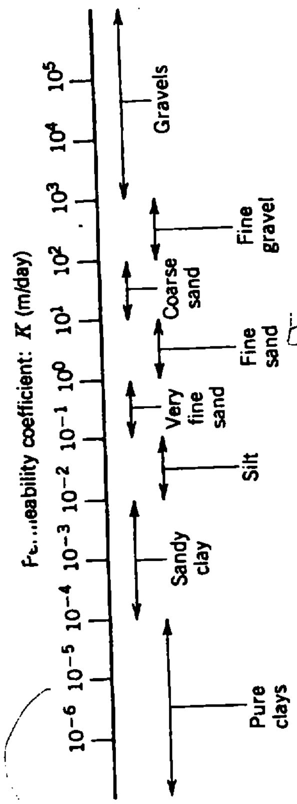


FIGURE 9.4 Range of permeability in soils.

Groundwater Quantity and Quality Control by Wanielista, Kersten, & Eaglin (1997)

HYDRAULIC CONDUCTIVITY

Hydraulic Conductivity was assumed to vary by location in the model based on influence from drainages, mud flats, or the Great Salt Lake. The distribution of hydraulic conductivity zones is shown on SHEET 12. Initial hydraulic conductivity values were chosen based on typical values for the types of materials encountered in the Kleinfelder borings. Soils consisted mostly of sands, silts, and clays. There were some gravels found near the mountains but these even had a silt & sand matrix. Wanielista, et. al. (1997) reports range of 0.3-30 ft/day for fine & coarse sands. An initial value of 7 ft/day was entered before calibration.

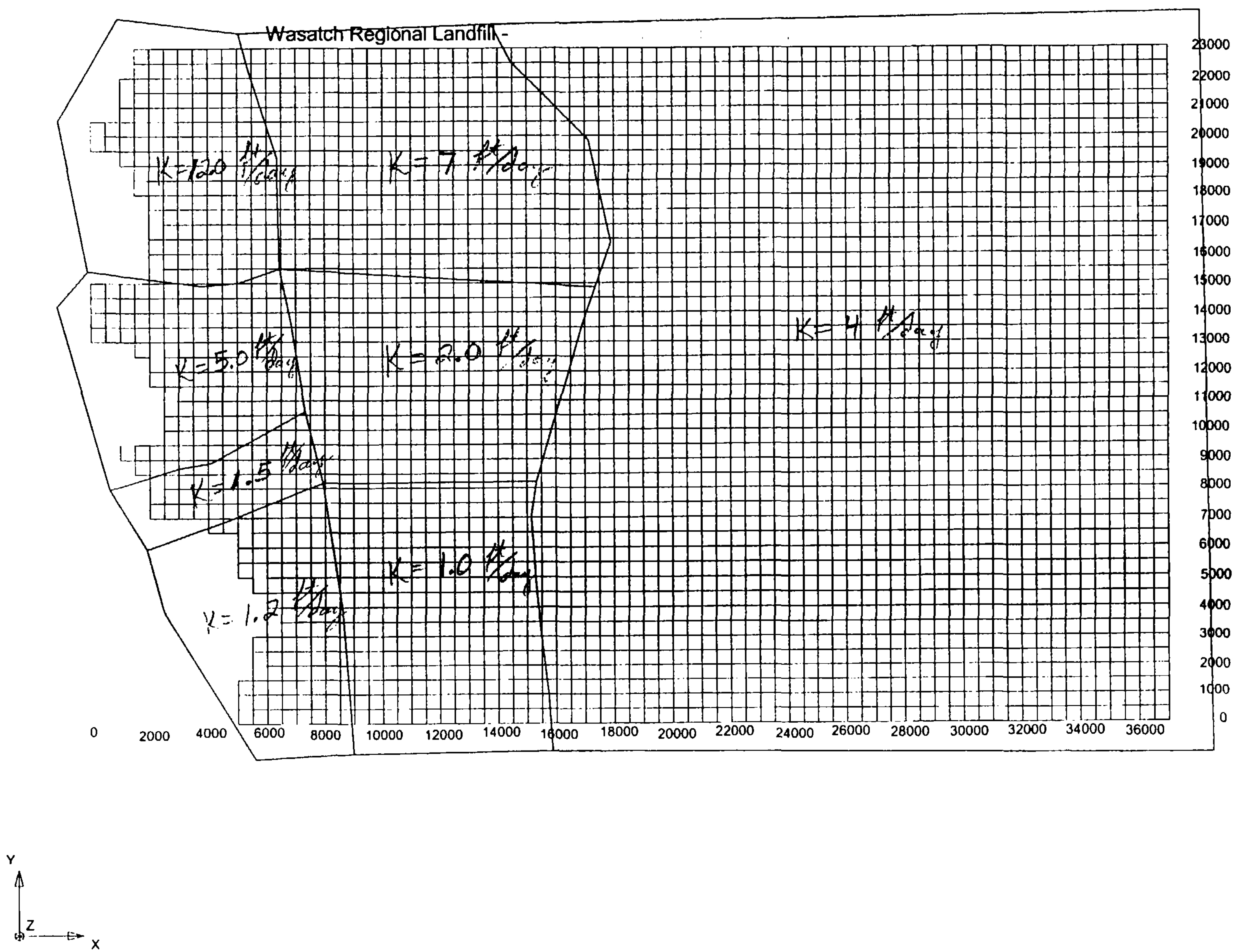
MODEL CALIBRATION

The hydraulic conductivity was varied to calibrate the computed groundwater levels to the measured groundwater levels from the borehole data assuming recharge & lake levels from 2003. The calibrated hydraulic conductivities are shown on sheet 12. Calibrated GW levels with calibration targets are on SHEET 13. Calibration targets show ± 3 feet within 95% confidence interval for computing standard deviation.

MODEL RESULTS

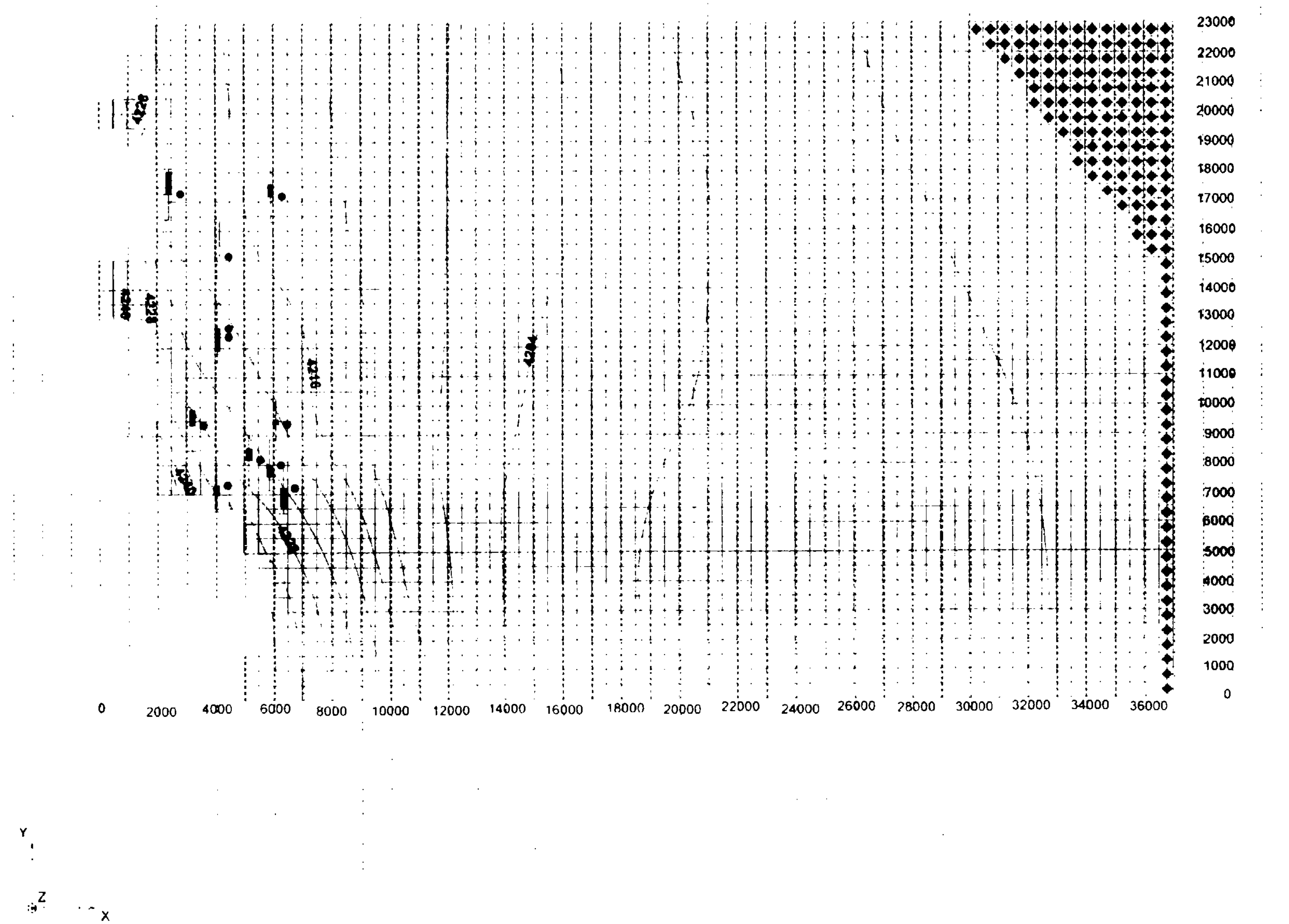
see SHEETS 14-16

The computed GW contours shown on Sheets 14-16 were overlain onto the landfill cell layout. Bottom elevations for the landfill were chosen a minimum of 6′ above the maximum projected computed groundwater level. [illegible]



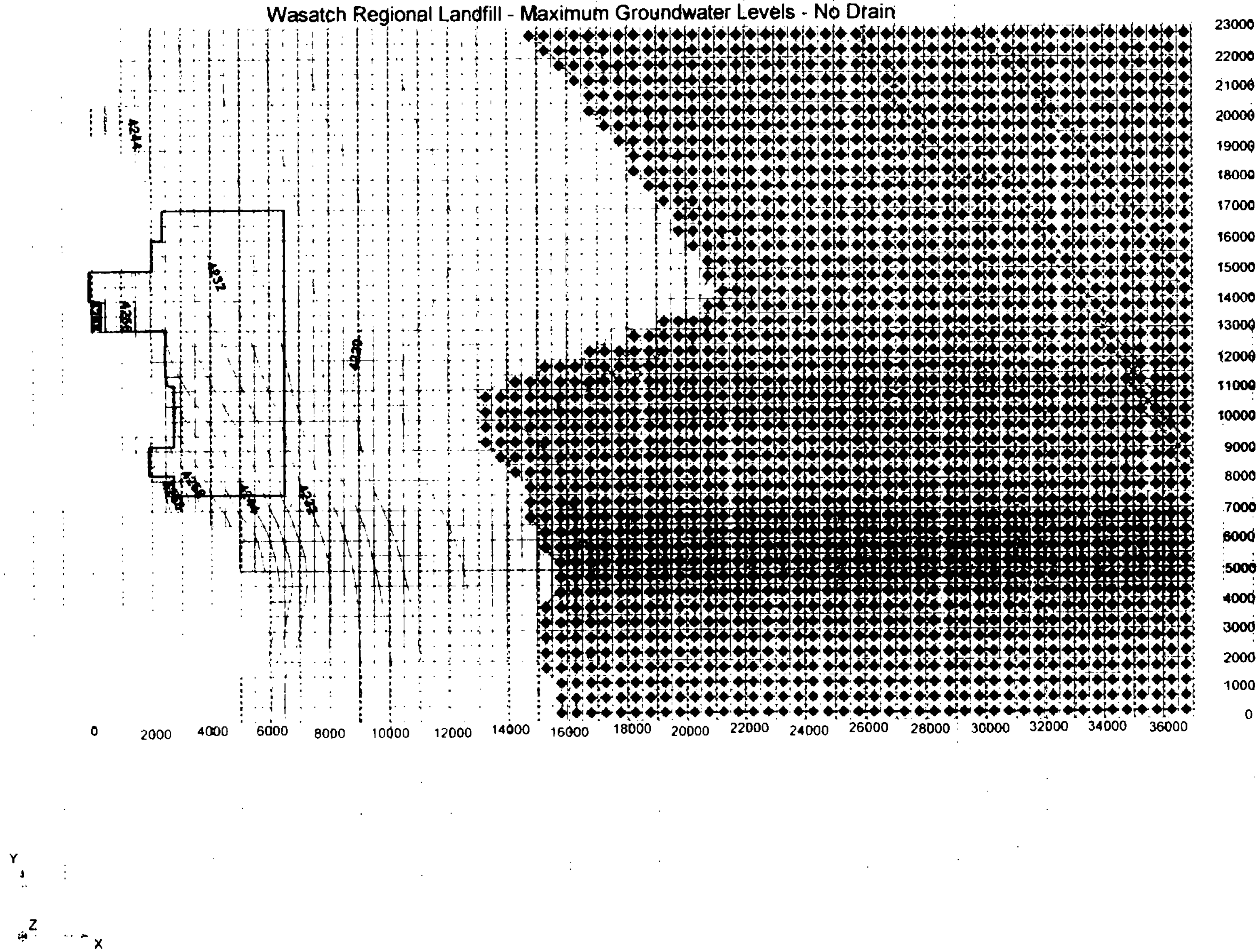
Wasatch Regional Landfill -
K=120 ft/day
K= 7 ft/day
K=5.0 ft/day
K=2.0 ft/day
K= 4 ft/day
K=1.5 ft/day
K= 1.0 ft/day
K= 1.2 ft/day
0 2000 4000 6000 8000 10000 12000 14000 16000 18000 20000 22000 24000 26000 28000 30000 32000 34000 36000
23000 22000 21000 20000 19000 18000 17000 16000 15000 14000 13000 12000 11000 10000 9000 8000 7000 6000 5000 4000 3000 2000 1000 0
Y
Z
X

12/17

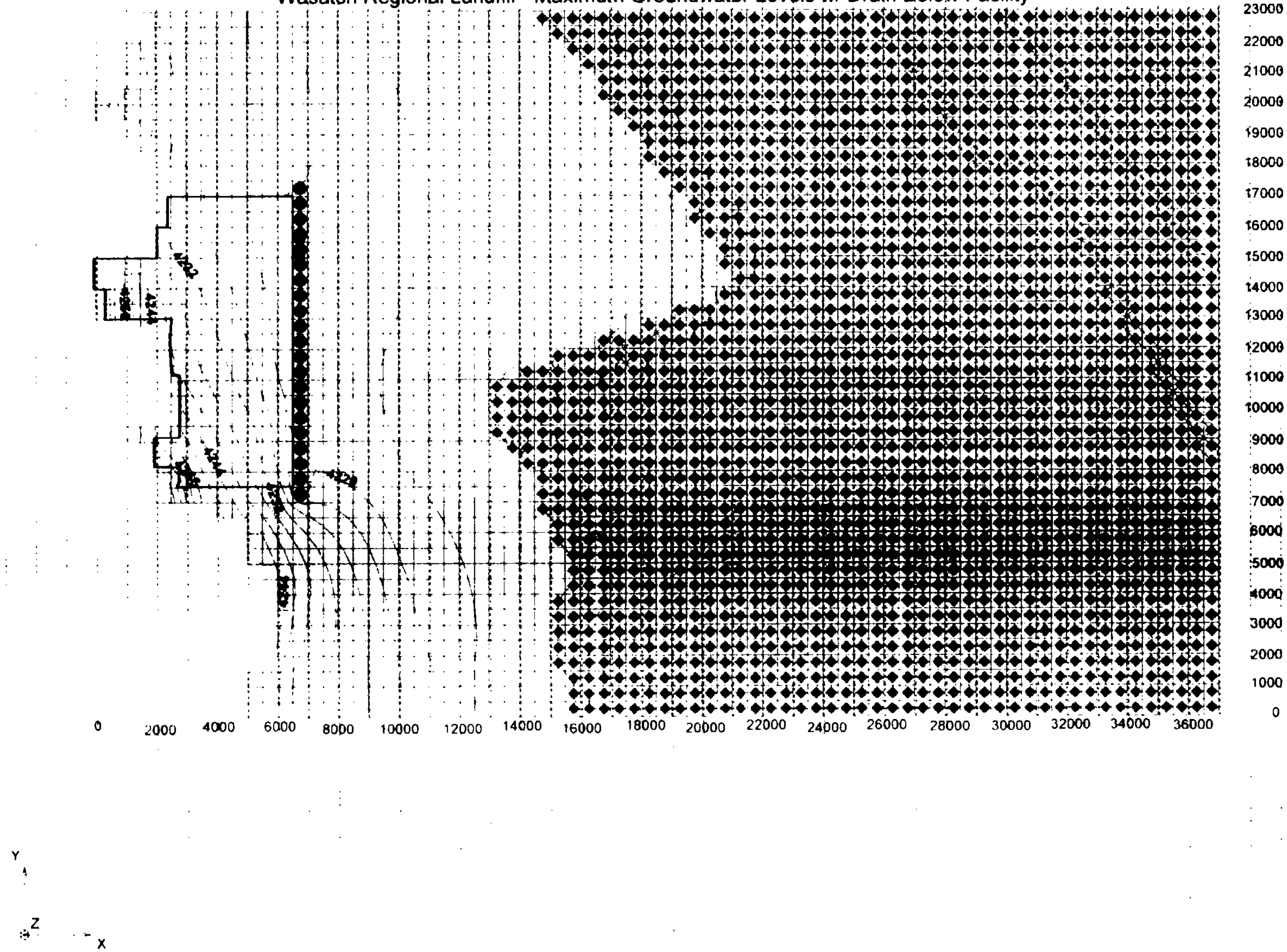


23000
22000
21000
20000
19000
18000
17000
16000
15000
14000
13000
12000
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
0
2000
4000
6000
8000
10000
12000
14000
16000
18000
20000
22000
24000
26000
28000
30000
32000
34000
36000
4204
4216
4228
Y
Z
X

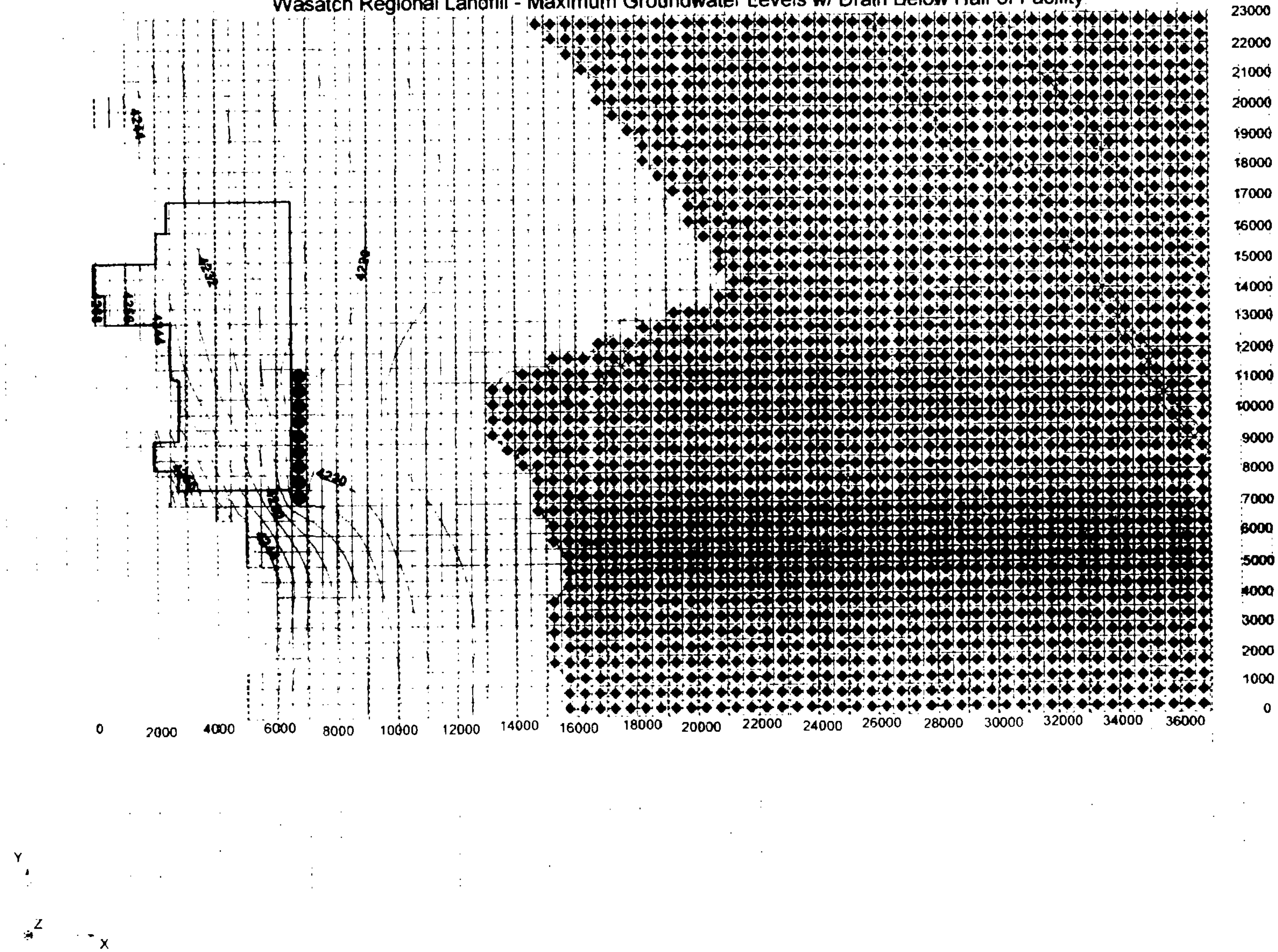
Wasatch Regional Landfill - Maximum Groundwater Levels - No Drain

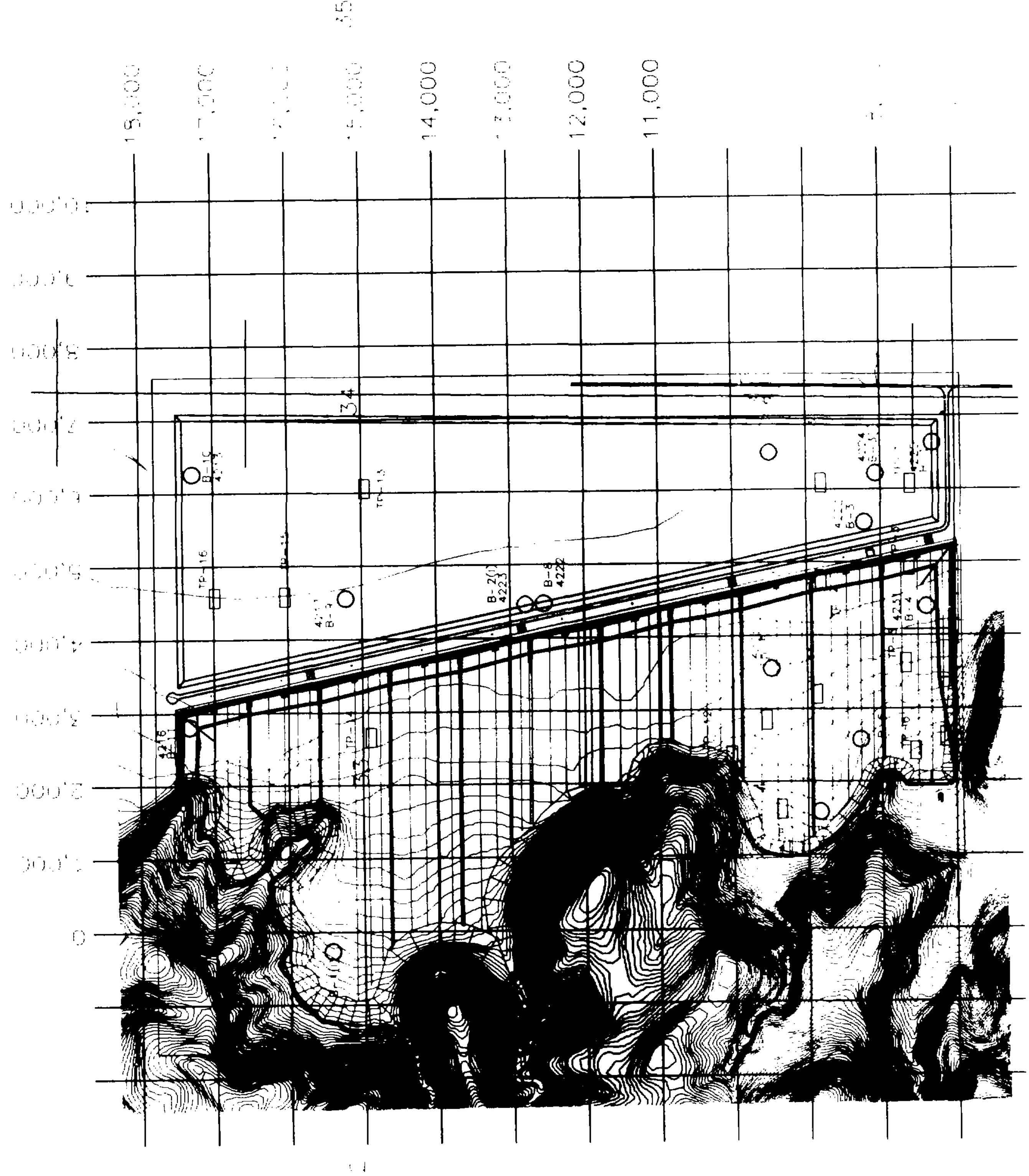


Wasatch Regional Landfill - Maximum Groundwater Levels w/ Drain Below Facility



Wasatch Regional Landfill - Maximum Groundwater Levels w/ Drain Below Half of Facility




14,000
12,000
11,000
0
B-8
4222
B-2
4223
TP-16
TP-15
4216

APPENDIX D

LANDFILL DESIGN CALCULATIONS

FLOOR ELEVATIONS

LEACHATE WITHDRAWAL PIPES

HYDROLOGIC EVALUATION OF LANDFILL PERFORMANCE (HELP) MODEL

LEACHATE COLLECTION SYSTEM

GEOTEXTILE FILTER FABRIC

SUMP CAPACITY

GCL HYDRAULIC COMPATIBILITY

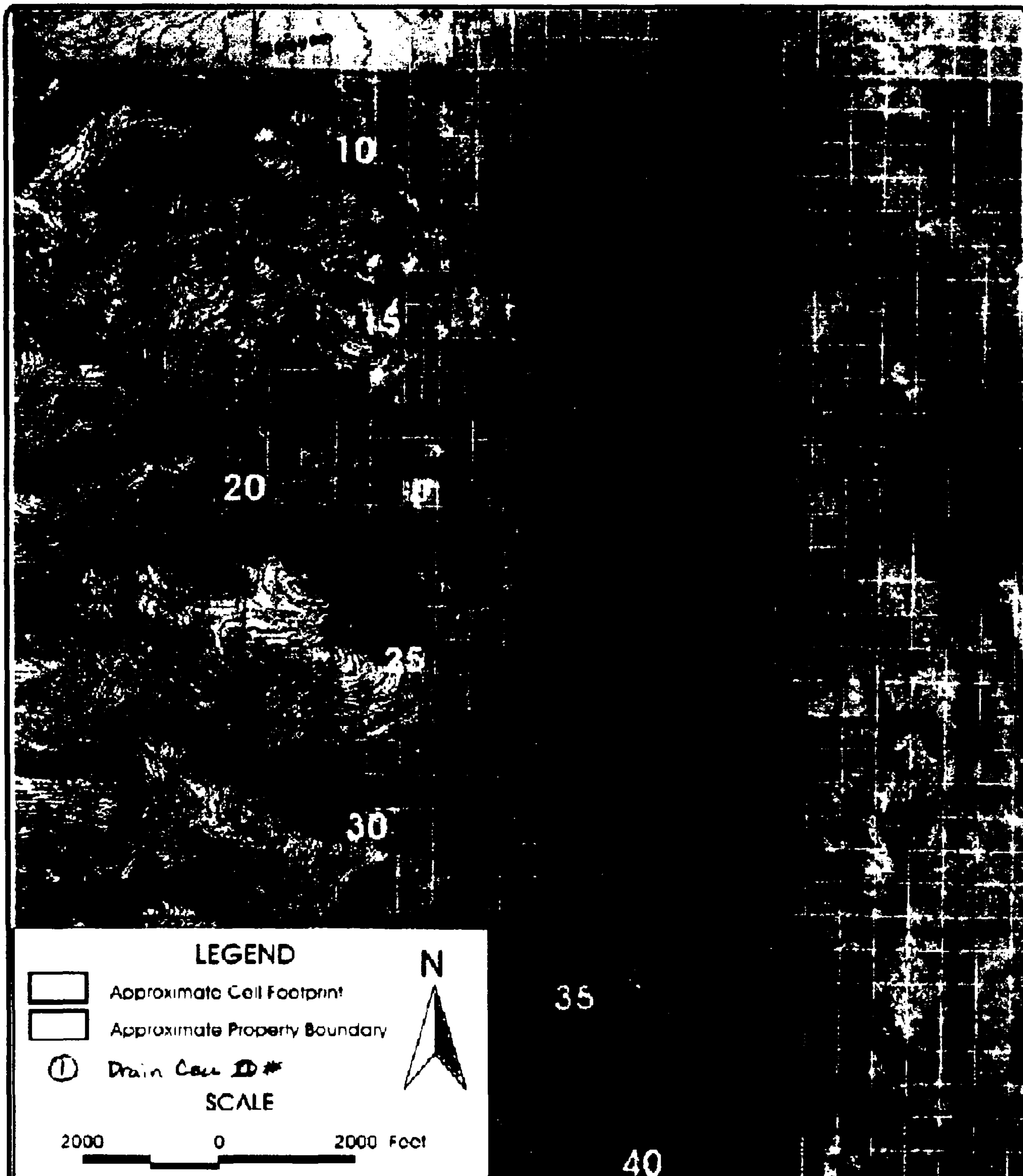
WASTE RUNOFF CONTAINMENT

HANSEN ALLEN & LUCE inc

CLIENT
PROJECT
FEATURE
PROJECT NO

SHEET 1 OF 5
COMPUTED
CHECKED
DATE

Drain Cell #	Column	Row	Top Width	Length in Cell	Area	Bed Length	Model K	Conductance
25	8	12	45	330	14850	2	12	80190
24	8	13	45	510	22950	2	12	123930
23	8	14	45	510	22950	2	12	123930
22	8	15	45	510	22950	2	12	123930
21	8	16	45	210	9450	2	12	51030
20	9	16	45	300	13500	2	12	72900
19	9	17	45	510	22950	2	5	51637.5
18	9	18	45	510	22950	2	5	51637.5
17	9	19	45	510	22950	2	5	51637.5
16	9	20	45	390	17550	2	5	39487.5
15	10	20	45	120	5400	2	5	12150
14	10	21	45	510	22950	2	5	51637.5
13	10	22	45	510	22950	2	5	51637.5
12	10	23	45	510	22950	2	5	51637.5
11	10	24	45	510	22950	2	5	51637.5
10	10	25	45	210	9450	2	5	21262.5
9	11	25	45	300	13500	2	5	30375
8	11	26	45	510	22950	2	5	51637.5
7	11	27	45	510	22950	2	5	51637.5
6	11	28	45	510	22950	2	1.5	15491.25
5	11	29	45	350	15750	2	1.5	10631.25
4	12	29	45	160	7200	2	1.5	4860
3	12	30	45	510	22950	2	1.5	15491.25
2	12	31	45	510	22950	2	1.5	15491.25
1	12	32	45	430	19350	2	1.2	10449



HANSEN ALLEN & LUCE INC

~~NEW CELL LAYOUT W/ BORROW~~

NEW DRAIN TRENCH LOCATION

FIGURE

??

HANSEN ALLEN & LUCE INC.

CLIENT

PROJECT

FEATURE

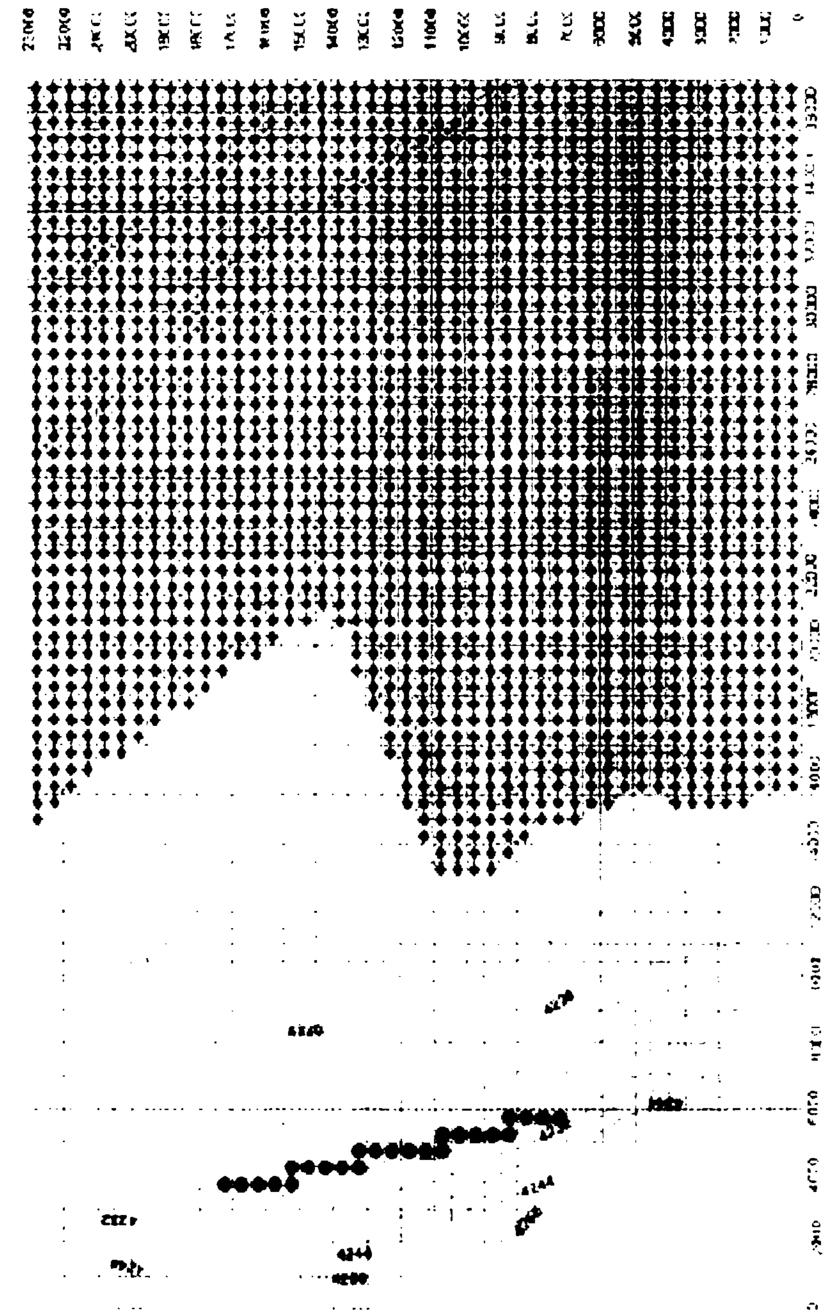
PROJECT NO

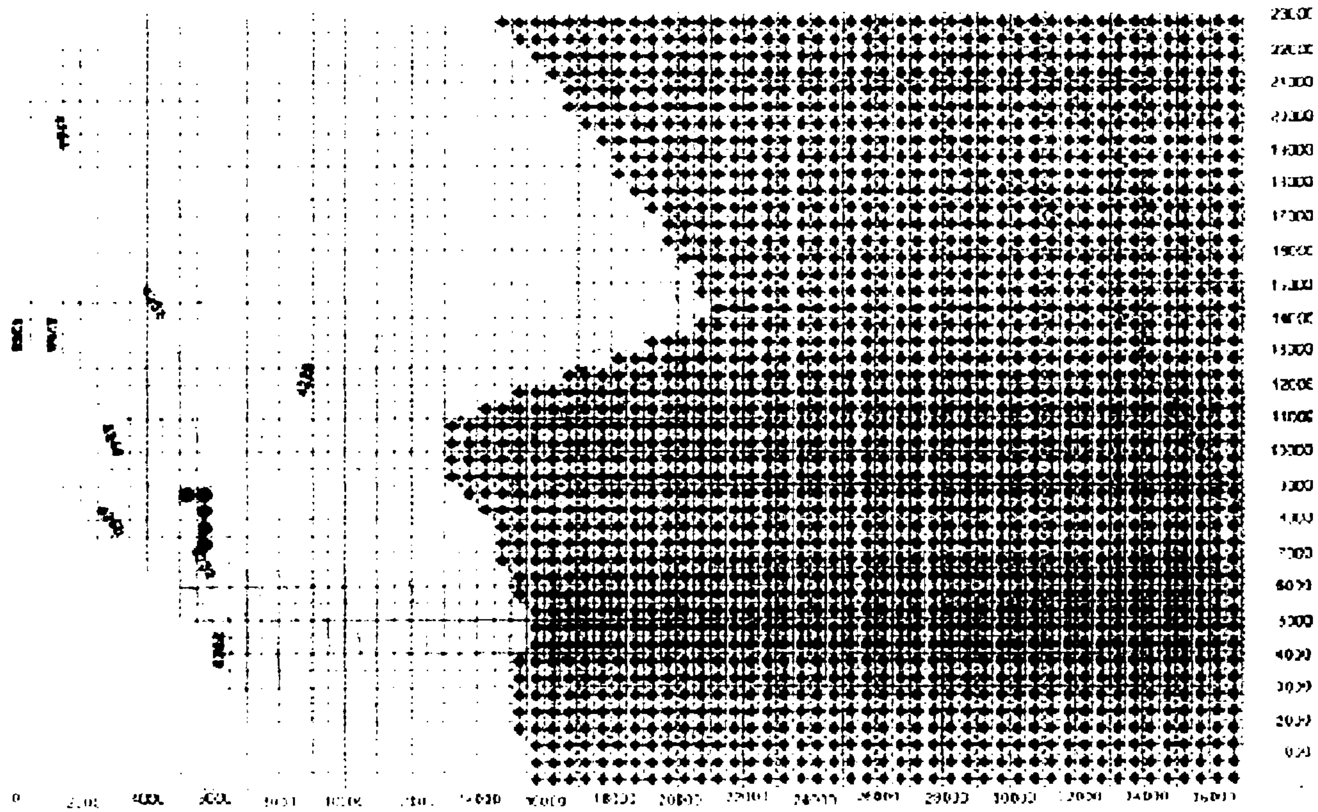
SHEET OF

COMPUTED

CHECKED

DATE





FLOOR ELEVATION

Client: ECDC Environmental
Project: Wasatch Regional Landfill
Feature: Floor Elevation Calculations
Date: December 2004, **REVISED JUNE 2005 (corrected and updated table - represents modified trench location and model)**

Description: Set the low point of each floor or leachate management area (phase) based on future groundwater projections and on potential settlement estimates.

Settlement: Assuming embankments approximately 15 feet high above existing ground surface, interior embankment slopes of 2H:1V, excavation to the cell floor of approximately 5 feet, and closure cap slopes of 4H:1V.

Horizontal distance to the floor from the top of the cell embankments is 20' x 2 = 40' from the top of the cell embankment to the low point of the phase area. Height of the closure cap above the embankment a the location of the low point of the sub-cell area is 40/4 = 10'. Total fill height above existing ground surface to the closure cap in the area of the sump is 15 + 10 = 25 feet.

If settlement is 3% of the fill height above existing grade, then 25 x 0.03 = 0.75 feet settlement. if the fill height increases to 30 feet above existing grade in the area of the sumps, then settlement is 30 x 0.03 = 0.90 feet.

Determine the low point elevation of each sub-cell area.

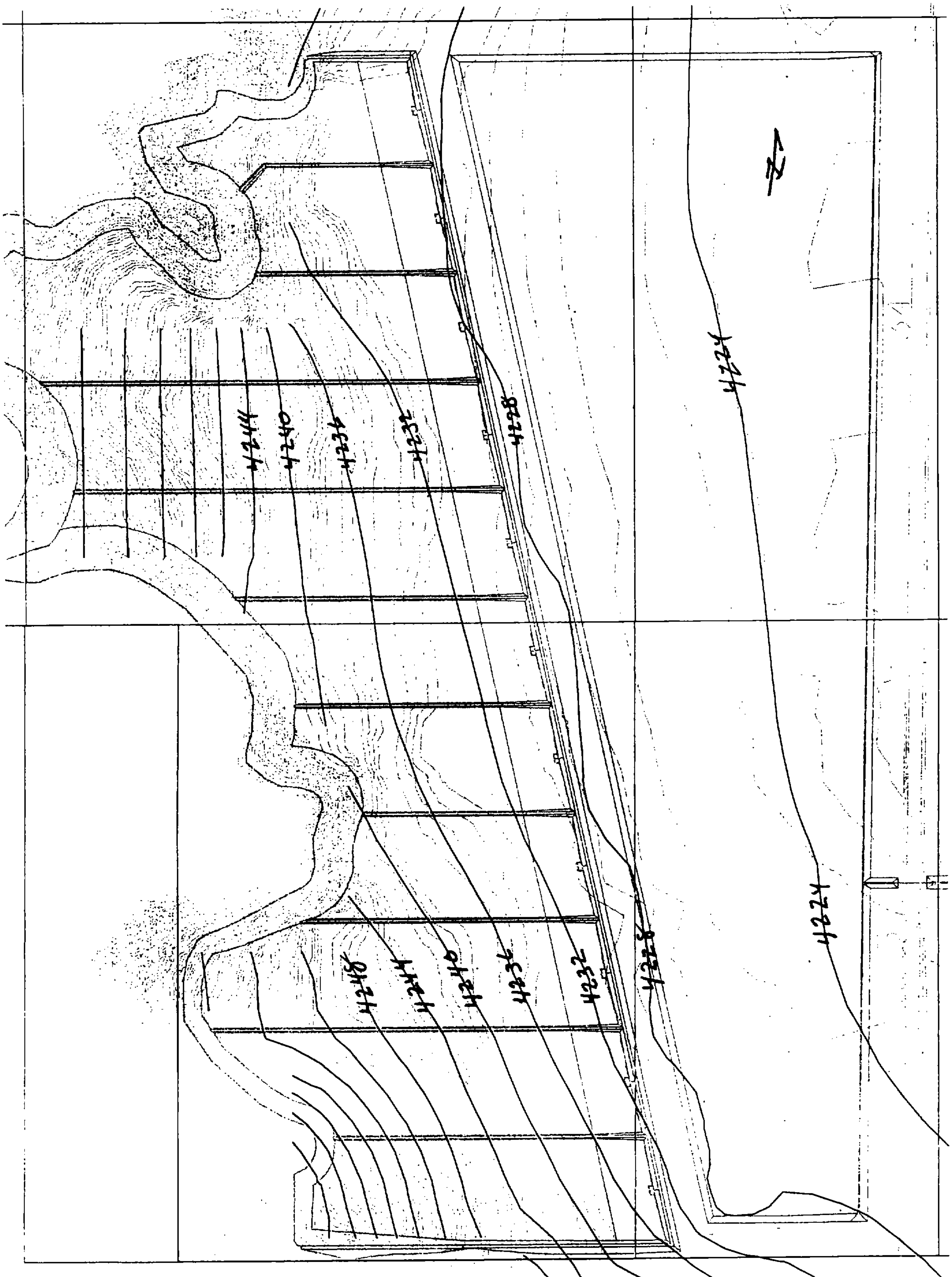
Provide a minimum ground water separation of the required 5 feet plus an additional foot for settlement and an additional 2.2 feet for modeling accuracy. Therefore, provide a minimum of 8.2 feet of separation.

Cell phases are designated as Phase 1 being the southmost phase and Phase 11 being the northmost phase.

Landfill Area					
Phase	**Ground Surface Elevation**	**Ground Water Elevation**	**Calc. Sump Potential Low point Elevations**	**Design Sump Low point Elevations**	**Separation to Projected High Ground Water**
1	4249.2	4233.6	4239.6	4243.5	9.9
2	4249.7	4231.6	4237.6	4243.5	11.9
3	4246.8	4230.8	4236.8	4243.5	12.7
4	4246.2	4230.0	4236.0	4243.5	13.5
5	4246.1	4229.2	4235.2	4243.5	14.3
6	4246.2	4229.5	4235.5	4243.5	14.0
7	4247.1	4229.0	4235.0	4243.5	14.5
8	4247.9	4229.1	4235.1	4243.5	14.4
9	4248.2	4228.1	4234.1	4243.5	15.4
10	4248.4	4228.1	4234.1	4243.5	15.4
11	4248.9	4228.7	4234.7	4243.5	14.8

Design the cell with all sump areas identical in configuration and elevation. The minimum design elevation for the sumps is 4241.8 to maintain 8.2 feet of separation between the sump liner system and the projected high ground water elevation.

The low points of the sumps are set at 4243.5 which provides a minimum separation of 9.9 feet to projected high ground water elevation.



Projected Ground Water Contours

1. Determine floor slopes required to maintain minimum slopes after accounting for potential differential settlement. Assume that the minimum planar slopes where geonet provides the drainage medium will be 2% after settlement and the minimum slopes for the leachate conveyance pipes will be 1% after settlement.

   a. Planar Slopes

      The worst case scenario for the planar slopes are those planes whose slopes are parallel to the slope of the closure caps. The floor slopes go up gradient toward the peak of the closure cap, thus, causing differential settlement that lessens the floor slope.

      Assuming a 100 foot wide sloping surface results in a rise of 2 feet on a 2% sloping floor surface. That same distance on the 4H:1V cap slope results on a rise of 25 feet. Therefore, the additional fill height for the waste pile and closure cap across the 100 foot wide surface is 25 feet resulting in a projected settlement amount of 0.50 foot to 0.75 foot at the up gradient side of the slope (2% to 3%). Adding an additional height of 0.50 foot to 0.75 foot to the 2 feet resulting from the 2% grade gives a resulting up gradient height of 2.5 feet to 2.75 feet. The resulting design slope should, therefore, be between 2.5% (2.5/100) and 2.75% (2.75/100). Design the slopes at 2.75%.

   b. Leachate Conveyance Pipe Slopes

      i. There are three different types of conditions to the leachate conveyance pipes on the cell floor. Pipes extend toward the west from the low point in the sumps to a point below the break line in the closure cap between the 4H:1V slope and the 5% cap slope; toward the west from the break line in the closure cap between the 4H:1V slope and the 5% cap slope to the west end of the cell; and pipes that extend along the inside toe of the east embankment slope. Each of the pipe configurations will be addressed separately.

         (1) Extending west from the low point in the sumps to a point below the break line in the closure cap between the 4H:1V slope and the 5% cap slope.

             These leachate conveyance pipes are located directly under the 4H:1V slope of the closure cap and their slopes are adversely effected by differential settlement.

             Assuming a 100 foot long length of pipe results in a rise of 1 foot on a 1% slope. That same distance on the 4H:1V cap slope results on a rise of 25 feet. Therefore, the additional fill height for the waste pile and closure cap along the 100 foot length of pipe is 25 feet resulting in a projected settlement amount of 0.50 foot to 0.75 foot at the up gradient side of the slope (2% to 3%). Adding an additional height of 0.50 foot to 0.75 foot to the 2 feet resulting from the 2% grade gives a resulting up gradient height of 1.5 feet to 1.75 feet. The resulting design slope should, therefore, be between 1.5% (1.5/100) and 1.75% (1.75/100). Design the slopes at 1.7%.

(2) Extending toward the west from the break line in the closure cap between the 4H:1V slope and the 5% cap slope to the west end of the cell.

Assuming a 100 foot long length of pipe results in a rise of 1 foot on a 1% slope. That same distance on the 5% cap slope results on a rise of 5 feet. Therefore, the additional fill height for the waste pile and closure cap along the 100 foot length of pipe is 5 feet resulting in a projected settlement amount of 0.10 foot to 0.15 foot at the up gradient side of the slope (2% to 3%). Adding an additional height of 0.10 foot to 0.15 foot to the 1 foot resulting from the 1% grade gives a resulting up gradient height of 1.1 feet to 1.15 feet. The resulting design slope should, therefore, be between 1.1% (1.1/100) and 1.15% (1.15/100). Design the slopes at 1.2%.

(3) Extend along the inside toe of the east embankment slope

Leachate collection pipes running parallel to the contour of the closure cap can be designed at a 1% slope since fill height does not increase along the length of the pipes and differential settlement is not projected to occur.

LEACHATE WITHDRAWAL

I. Evaluate the long-term strength of the HDPE pipe against failure or significant loss of cross-sectional area.

Reference Manuals: "Design & Engineering Guide for Polyethylene Piping", by Rinker Materials, August 2003.

"Plexco/Spirolite Engineering Manual 2. System Design", by Chevron Chemical Co., April 1996.

Design Criteria:

Pipe Diameters = 24 inches - top and bottom pipes.
Maximum Design Height of Overburden = 250 feet (See attached drawing)
Note: Maximum height of overburden on the design drawing is 235.8 feet. However a larger design height was selected to account for uncertainties in the construction and filling of the landfill, as well as additional load applied by the operation equipment over the landfill.

Unit weight of overburden:

Soil cover	= 125	pcf
Waste	= 80	pcf

A. <u>Soil Pressure by components</u>

$$P_T = P_S + P_L$$

where: P_T = Total load pressure
P_S = Static or dead load pressure
P_L = Live load pressure

Using the Boussinesq's Equation from the manual reference above, the live load pressure can be estimated as follows

$$P_L = \frac{3W_L H^3}{2\pi * R^5}$$

W_L = wheel load (lb)
H = vertical depth of crown
R = distance from the point load application to the crown

Assuming a tire load of 4,000 pound, then the live load on the pipe would be as follows

$$P_L = \frac{3(4000)(250)^3}{2\pi *(250)^5}$$

P_L = 0.03 psf (load is insignificant to the dead load and will be excluded)

Therefore, only the dead load will be used to pipe strength design.

P_T = P_S = height of overburden x unit weight of overburden
$P_{T24"}$ = (2' + 2' + 3')(125 pcf) ÷ (95')(80 pcf) + (10')(62.4)

= 9,099 psf = 63.2 psi for the 24" pipe

$P_{T16"}$ = (2' + 2' + 3')(125 pcf) + (91')(80 pcf) + (10')(62.4)
= 8,779 psf = 61.0 psi for the 16" pipe

B. Evaluate Wall Crushing

The compression stress on the pipe walls is given below:

$$S = \frac{P_L D_O}{288t}$$

S = Compressive stress (psi)
P_L = vertical load applied to pipe (psf)
t = wall thickness (in)
D_O = outside diameter of pipe (in)

The maximum long-term design stress value for Plexco polyethylene pipe is 800 psi. The ratio of pipe diameter to wall thickness is given below.

$$\frac{D_O}{t} = \frac{288(800)}{9{,}099\,psf}$$

$$\frac{D_O}{t} = 25.3$$

Therefore a SDR of 25 or lower should be strong enough to avoid crushing failure.

C. Evaluate Wall Buckling

Wall buckling resistance of pipe is increased when it is buried. The soil and pipe work together to resist buckling. AWWA C-950 gives a design equation for buckling of buried plastic pipe which is applicable to PLEXCO pipe.

$$P_{ch} = \frac{1}{SF}\sqrt{\left(\frac{2.67 \cdot R_w \cdot B \cdot E_s \cdot E}{DR^3}\right)}$$

P_{cb} = Critical buckling stress (psi)
SF = Safety factor,
R_w = Water buoyancy factor, (dimensionless)
B = Empirical Coefficient of Elastic Support (dimensionless)
E_s = Soil modulus, (See Table C-4)
E = Pipe modulus of elasticity, psi
DR = Dimension ratio

Where,

$$R = 1 - \left(0.33 \cdot \frac{H_w}{H}\right)$$

H_w = Height of water table above the pipe (ft)
The embankment is 10 ft high, so the maximum water height will be 10 ft
H = Height of soil cover above pipe (ft)
The cover over the sump area is about 102 ft

$$B = \frac{1}{1 + 4e^{(-0.065H)}}$$

e = Natural log base number
H = Height of soil cover above pipe (ft)

For the 24" pipe:

$$P_{cb} = \frac{1}{2}\sqrt{\frac{2.67 \cdot (0.968) \cdot (0.995) \cdot (30{,}000psi)(1600psi)}{(15.5)^3}}$$

$$P_{cb} = 91.0psi$$

$$R = 1 - 0.33\frac{10'}{102}$$

$$R = 0.968$$

$$B = \frac{1}{1 + 4e^{(-0.065(102))}}$$

$$B = 0.995$$

The pipe should not buckle since the calculated buckling resistance of 91.0 psi exceeds the 63.2 psi loading on pipe.

For the 16" pipe:

$$P_{cb} = \frac{1}{2}\sqrt{\frac{2.67 \cdot (0.966) \cdot (0.993) \cdot (30{,}000psi)(1000psi)}{(15.5)^3}}$$

$$P_{cb} = 71.8psi$$

$$R = 1 - 0.33\frac{10'}{98}$$

$$R = 0.966$$

$$B = \frac{1}{1 + 4e^{(-0.065(98))}}$$

$$B = 0.993$$

The pipe should not buckle since the calculated buckling resistance of 71.8 psi exceeds the 61.0 psi loading on pipe.

D. Evaluate Ring Deflection

Ring deflections are calculated using the following modified Spangler's equation:

$$\Delta X = \frac{D_l \cdot K \cdot W}{\left(\frac{2E}{3(DR-1)^3}\right) + 0.061E'}$$

ΔX = Horizontal deflection (in.)
D_l = Deflection lag factor, PolyPipe recommends 1.0 (dimensionless)
K = Bedding constant, Polypipe recommends 0.1 (dimensionless)
W = Earthload (lbs/inch)
E = Modulus of elasticity of pipe, 30,000 psi
E' = Soil modulus
DR = Dimension ratio

For the 24" pipe:

$$\Delta X = \frac{1 \cdot 0.1 \cdot (63.2 \cdot 24)}{\left(\frac{2 \cdot 30{,}000}{3(15.5-1)^3}\right) + 0.061 \cdot 1600}$$

$$\Delta X = 1.46 in$$

The percent deflection is calculated using the following formula:

$$d = \frac{\Delta X}{D} \cdot 100$$

d = Percent deflection (%)
ΔX = Horizontal deflection (in.)
D = Outside diameter (in.)

$$d = \frac{1.46}{24} \cdot 100$$
$$d = 6.07\%$$

To see if this deflection could cause failure in the pipe the ring bending strain was computed below. This equation is provided in the Plexco/Spirolite Engineering Manual.

$$\varepsilon = f_D \frac{\Delta Y}{D_M} \frac{2C}{D_M}$$

C = 0.53t = 0.53 *1.548 = 0.82

ε = wall strain, (%)
f_d = deformation shape factor
D_M = mean diameter (in)
C = outer fiber to wall centroid (in)
t = pipe minimum wall thickness

$$\varepsilon = 6\frac{1.46}{22.36}\frac{2(0.82)}{22.36}$$
$$\varepsilon = 0.0287 = 2.87\%$$

The PLEXCO design manual references a study by Jansen that states strains of 8% should perform well for at least 50 years. ISCO industries also lists its high density polyethylene pipe as having an elongation at yield of 8%.

For the 16" pipe:

$$\Delta X = \frac{1 \cdot 0.1 \cdot (61.0 \cdot 16)}{\left(\frac{2 \cdot 30{,}000}{3(15.5 - 1)^3}\right) + 0.061 \cdot 1000}$$

$$\Delta X = 1.44in$$

The percent deflection is calculated using the following formula:

$$d = \frac{\Delta X}{D} \cdot 100$$

d = Percent deflection (%)
ΔX = Horizontal deflection (in.)
D = Outside diameter (in.)

$$d = \frac{1.44}{16} \cdot 100$$
$$d = 9.03\%$$

To see if this deflection could cause failure in the pipe the ring bending strain was computed below. This equation is provided in the Plexco/Spirolite Engineering Manual.

$$\varepsilon = f_D \frac{\Delta Y}{D_M}\frac{2C}{D_M}$$

C = 0.53t = 0.53 *1.032= 0.547

ε = wall strain, (%)

f_d = deformation shape factor
D_M = mean diameter (in)
C = outer fiber to wall centroid (in)
t = pipe minimum wall thickness

$$\varepsilon = 6\frac{1.44}{14.91}\frac{2(0.547)}{14.91}$$
$$\varepsilon = 0.0425 = 4.25\%$$

The PLEXCO design manual references a study by Jansen that states strains of 8% should perform well for at least 50 years. ISCO industries also lists its high density polyethylene pipe as having an elongation at yield of 8%.

II. Check the required length of HDPE pipe to allow for contraction/expansion due to thermal changes.

A. Differential Pipe Length Due to Temperature Changes

The bottom pipes will be backfilled and therefore not exposed to extreme temperature fluctuations. However the top pipe will be exposed during construction and may experience large temperature variations.

Assume maximum $\Delta T = 100° - 10° = 90°$
$$\Delta L = \alpha \times \Delta T \times L$$

$$L = 21.2'$$

α = coefficient of thermal expansion
$\alpha = 1.0 \times 10^{-4}$ in/in/°F
L = pipe length in feet

ΔL =(1.0 x 10^{-4} in/in/°F)(90°F)(15')(12 in/ft) = 1.62 in. = 0.135 ft.

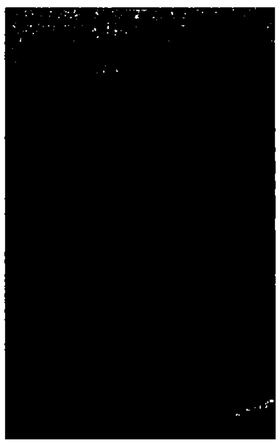
Only approximately 15' of the top of the pipe will be exposed to the thermal fluctuations assumed above. This amount of expansion and contraction is well within the 8% discussed previously.

High Density Polyethylene

TYPICAL PROPERTIES

HIGH DENSITY POLYETHYLENE PIPE
Typical Physical Properties***

Property	Specification	Unit	Nominal Value
Material Designation	PPI / ASTM		PE 3408
Material Classification	ASTM D-1248		III C 5 P34
Cell Classification	ASTM D3350-99		345464C
-Density (3)	ASTM D-1505	gm/cm3	0.955
-Melt Index (4)	ASTM D-1238 (216 kg/190iC)	gm/10 min.	0.11*
-Flex Modulus (5)	ASTM D-790	psi	135,000
-Tensile Strength (4)	ASTM D-638	psi	3,200
PENT (6)	ASTM F-1473	Hours	>100
-HDB @73i F (4)	ASTM D-2837	psi	1,600
-HDB @ 140 Deg F	ASTM D-2837	psi	800
-U-V Stabilizer (C)	ASTM D-1603	% C	2.5
Hardness	ASTM D-2240	Shore "D"	65
Compressive Strength (yield)	ASTM D-695	psi	1,600
Tensile Strength @ Yield (Type IV Spec.)	ASTM D-638 (2"/min.)	psi	3,200
Elongation @ Yield	ASTM D-638	%, minimum	8
Tensile Strength @ Break (Type IV Spec.)	ASTM D-638	psi	5,000
Elongation @ Break	ASTM D-638	%, minimum	750
Modulus of Elasticity	ASTM D-638	psi	130,000



PENT (6)	ASTM F-1473	Hours	>100
(Cond. A, B, C: Mold. Slab)	ASTM D-1693	Fo, Hours	>5,000
(Compressed Ring - pipe)	ASTM F-1248	Fo, Hours	>3,500
Slow Crack Growth	Battelle Method	Days to Failure	>64
Impact Strength (IZOD) (.125Ó Thick)	ASTM D-256 (Method A)	In-lb / in notch	42
Linear Thermal Expansion Coef.	ASTM D-696	in / in/iF	1.2x10-4
Thermal Conductivity	ASTM D-177	BTU-in/ft2/ hrs/ degreesF	2.7
Brittleness Temp.	ASTM D-746	degrees F	< -180
Vicat Soft. Temp.	ASTM D-1525	degrees F	257
Heat Fusion Cond.	ASTM D-1525	@ psi degrees F	75 @ 400

*** This list of typical physical properties is intended for basic characterization of the material and does not represent specific determinations of specifications. The physical properties values reported herein were determined on compression molded specimens prepared in accordance with Procedure C of ASTM D 1928 and may differ from specimens taken from pipe.
** Tests were discontinued because no failures and no indication of stress crackinitiation.
* Average Melt Index value with a standard deviation of 0.01

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Pipe Stiffness

for

Buried Gravity Flow Pipes

TN-19/2000

Foreword

This report was developed and published with the technical help and financial support of the members of the PPI (Plastics Pipe Institute, Inc). The members have shown their interest in quality products by assisting independent standards-making and user organizations in the development of standards, and also by developing reports on an industry-wide basis to help engineers, code officials, specifying groups, and users.

The purpose of this technical note is to provide general information on pipe stiffness for buried, gravity flow pipes.

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April, 2000

PIPE STIFFNESS FOR BURIED GRAVITY FLOW PIPES

Various measures have been used to characterize the ring bending stiffness of pipe. In the U.S., these measures include:

- Flexibility Factor (FF) as defined in AASHTO Bridge Design Specification Section 18,
- Pipe Stiffness (PS) as defined in ASTM D 2412, and
- Ring Stiffness Constant (RSC) as defined in ASTM F 894.

These measures characterize the pipe's resistance to ring deflection when subjected to a short-term parallel plate load. The purpose of this note is to advise on the applicability of these measures for comparing and classifying plastic pipes.

The first commonly used measure for pipe deflection resistance was pipe stiffness (PS). Designers found it easy to assign a minimum PS value in their specifications for plastic pipes. However, for larger diameter pipes, the validity of PS as a product specification requirement has been questioned because:

(1) It was discovered that given the same handling and installation forces smaller diameter pipes require much higher stiffness for proper installation than do larger diameter pipes.

(2) It was found that there was a trade-off between pipe material strain capacity and pipe stiffness. Pipes made from strain limited plastics such as glass-reinforced thermoset resin required greater stiffness to restrain localized deflections than that required for thermoplastic pipes.

HANDLING AND INSTALLATION

Pipe intended for buried applications must be sufficiently stiff to resist deflection due to shipping, handling, and storage loads as well as the loads applied during installation. The most significant of these loads is the force exerted on the pipe during mechanical compaction of the soil. This force can cause the pipe to undergo deformations that will be exacerbated by soil loads during the subsequent placement of backfill. The force exerted on the pipe during compaction can be treated as a line load that is primarily a function of the compaction method and soil type and is relatively independent of the pipe's diameter.

When pipes of equal PS but different diameters are subject to equal line loads, the deflection response in percent is a function of its diameter. For a given line load, the deflection of a pipe can be calculated from the PS equation:

$$PS = \frac{F}{\Delta Y} = \frac{EI}{.149\, r_m^3} \qquad (1)$$

Where:	F	=	Load (lbs./lineal-in)
	ÄY	=	Deflection (in)
	E	=	Modulus of Elasticity (psi)
	I	=	Cross Sectional Moment of Inertia (in^4/in)
	r_m	=	Mean Radius (in)

The difficulty encountered when trying to classify pipes of different diameters using PS can be seen by comparing the deflection response of 12" pipe with a 60" pipe both having a PS of 50 psi and both subjected to a 50 lbs/lineal-in parallel plate load. Both pipes will deflect one inch per Eq. 1. However, when deflection is calculated in percentage as it normally is for buried pipes, the 12" pipe deflects 8.3 percent of its initial diameter while the 60" pipe deflects only 1.7 percent. From this, the conclusion can be drawn that PS is not very useful for classifying pipes of different diameters in regard to installation forces. Given the same handling and installation forces it can be seen that smaller diameter pipes require more PS than larger diameter pipes.

The above discussion leads to the conclusion that any workable minimum stiffness requirement has to be diameter weighted. This can be done by "weighting" the PS equation. The PS equation can be weighted by multiplying both sides of Eq. 1 by the mean diameter. The result of this multiplication, after rearranging terms is given in Eq. 2.

$$\frac{F}{\frac{\Delta Y}{D_m}} = \frac{8EI}{.149\, D_m^3} \qquad (2)$$

If the load in Eq. 2 is expressed in lbs/ft instead of lbs/in and if deflection is expressed in units of percent, Eq. 2 becomes:

$$RSC = \frac{F}{\frac{\Delta Y}{D_m}}\left(\frac{12}{100}\right) = \frac{6.44EI}{D_m^3} \qquad (3)$$

Eq. 3 is the mathematical expression of RSC. It can be shown that subjecting a 12" pipe and a 60" pipe of equal RSC to an equal parallel plate load would produce an equal percent deflection. The FF is merely the inverse of the RSC multiplied by a constant. Therefore, both the FF and RSC produce equal deflection responses and can be used to classify pipes.

What minimum value of RSC is necessary to provide sufficient resistance to handling and installation forces? ASTM F 894 anticipates up to 3 percent out-of-roundness for pipe prior to earthloading. Therefore, the pipe should be able to withstand normal handling and installation loads, such as the force transmitted to the pipe due to machine compaction of the embedment, without exceeding 3 percent out-of-roundness. (This is not to be confused with the deflection limit applied to deflections due to backfill and live loads.) Field measurements reported by Petroff [1] show that HDPE pipes with RSC of 40 possess sufficient stiffness to resist normal handling and installation loading and remain within 3 percent out-of-roundness when installed in accordance with ASTM D 2321 or PPI TR-31.

It should be noted that the ASTM test methods for RSC and PS differ. The RSC test is done at a load rate of 2 in/min as opposed to 0.5 in/min for PS. And, RSC is measured at 3.0 percent deflection whereas PS is measured at 5.0 percent. Because of these differences when the expression in Eq. 3 is used to convert from RSC to PS, the F/ÄY value given by Eq. 3 should be multiplied by an empirical factor for HDPE of 0.8. (This factor can vary with material.)

This section has shown that as the diameter of a pipe increases, less stiffness is required to achieve the same capacity for handling and installation. For instance, a 72" pipe with a tested RSC of 40 would have a PS of 4.6 psi. This PS may seem low, but the RSC is sufficient for handling and installation. However, a PS of 4.6 psi would typically be insufficient for a small diameter pipe. Consider a 6" pipe with the same PS (4.6 psi). It would have an RSC of 4.2, which is far below the minimum 40 required for proper installation. As a matter of fact a 6" pipe having a 46-psi stiffness would have an RSC of 41.4. So, the minimum RSC requirement of 40 is consistent with the early experience of the plastic pipe manufacturers in that a relatively high stiffness was required for proper installation.

STRAIN CAPACITY

When designing buried applications, the designer can make a trade-off between the strain capacity of the pipe material and the pipe's stiffness. When subjected to earth loads, strain occurs in the pipe wall as a result of deformations due to both ring bending and ring thrust. If a pipe material has a low tolerance for strain, it is usually necessary to limit the strain by limiting the pipe deformation. There are two levels of deformation in a buried pipe. One is standard diametrical deflection due to earth load; the other is a second order deformation due to non-elliptical deformation. Second order deformations are small but may induce high strains. They are directly proportional to the pipe's ring stiffness. These deformations are of little consequence with HDPE pipes, because of the high strain capacity. Janson recently completed an eight-year study on pressure-rated grade HDPE and reports that for practical design purposes (for gravity sewers) there does not seem to be an upper limit on design strain [2]. This essentially means that when using pressure-rated grades of HDPE, a designer does not have to be concerned with the strains occurring from second order deformations, assuming overall deflection and buckling are controlled.

BURIED PERFORMANCE

Buried pipe must possess sufficient stiffness to mobilize soil resistance in the backfill and to resist buckling. Deflection must be limited to a value that will not disrupt flow or cause joint leakage. The considerable field experience with stress-rated HDPE pipes of high SDR's and over 25 years experience with stress-rated HDPE, profile wall pipes speaks to the capability of low stiffness pipes to perform under soil loads.

Flexible pipe deflection depends on the combined contribution of pipe ring bending stiffness and embedment soil stiffness (E'). Considerable testing and field measurements have established that for low stiffness pipes the deflection is virtually controlled by the embedment soil. This is true for any flexible pipe, whether metal or plastic. Spangler's Iowa formula can be used to demonstrate that the soil's contribution to resisting deflection is much more significant than the pipe's contribution. Although Spangler's equation was developed using pipes of 25-psi stiffness and higher, considerable field experience has shown its applicability to low stiffness pipes [3]. When pipes of 46 psi PS and, say, 4.6 psi PS are installed with E's normally associated with pipe installations, there is little difference in their deflection. On the other hand when pipe is not installed properly a low E' results in both the 46 psi and 4.6 psi deflecting excessively. It can be shown mathematically that a 46 psi pipe supplies a stiffness to the soil/pipe system equivalent to a soil with an E' of 112 which offers hardly any resistance to deflection. Therefore, whether the PS is 46 psi or 4.6 psi as in the example above, the soil placement will control deflection.

The principle of soil embedment controlling deflection has been illustrated over and over again in field tests and numerous soil box demonstrations. For instance, one soil box test conducted at Utah State University on a 21" HDPE pipe with a stiffness of 6.4 psi installed in silty sand at 92 percent of Standard Proctor density resulted in 3 percent deflection with a loading equivalent to 90 feet of soil backfill.

Publications by Chua and Lytton [4], Watkins et al [5], Gaube and Muller [6], Taprogge [7], Janson and Molin [8], Selig [9], and Gabriel [10] all speak to the fact that the pipe's stiffness makes only a minimal contribution to deflection resistance.

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Table A-2 (cont)
PIPE WEIGHTS AND DIMENSIONS *(IPS)*
PE3408 *(BLACK)*

OD Nominal in.	OD Actual in.	OD Actual mm.	SDR	Nominal ID in.	Nominal ID mm.	Minimum Wall in.	Minimum Wall mm.	Weight lb. per foot	Weight kg. per meter
			9	18.45	468.71	2.667	67.73	77.845	115.847
			9.3	18.63	473.26	2.581	65.55	75.658	112.592
			11	19.46	494.33	2.182	55.42	65.237	97.084
			11.5	19.66	499.34	2.087	53.01	62.690	93.294
24	24.000	609.60	13.5	20.30	515.68	1.778	45.16	54.206	80.668
			15.5	20.78	527.80	1.548	39.33	47.731	71.032
			17	21.06	535.01	1.412	35.86	43.801	65.184
			21	21.62	549.22	1.143	29.03	35.907	53.436
			26	22.08	560.83	0.923	23.45	29.299	43.601
			32.5	22.46	570.59	0.738	18.76	23.638	35.177
			11	22.71	576.72	2.545	64.65	88.795	132.142
			11.5	22.94	582.57	2.435	61.84	85.329	126.983
			13.5	23.69	601.62	2.074	52.68	73.781	109.798
			15.5	24.24	615.76	1.806	45.88	64.967	96.682
28	28.000	711.20	17	24.57	624.18	1.647	41.84	59.618	88.722
			21	25.23	640.76	1.333	33.87	48.874	72.732
			26	25.76	654.30	1.077	27.35	39.879	59.346
			32.5	26.21	665.68	0.862	21.88	32.174	47.880
			11	24.33	617.91	2.727	69.27	101.934	151.694
			11.5	24.57	624.18	2.609	66.26	97.954	145.771
			13.5	25.38	644.60	2.222	56.44	84.697	126.043
			15.5	25.97	659.74	1.935	49.16	74.580	110.987
30	30.000	762.00	17	26.33	668.77	1.765	44.82	68.439	101.849
			21	27.03	686.53	1.429	36.29	56.105	83.494
			26	27.60	701.04	1.154	29.31	45.779	68.127
			32.5	28.08	713.23	0.923	23.45	36.934	54.965
			13.5	27.07	687.57	2.370	60.21	96.367	143.409
			15.5	27.71	703.73	2.065	52.44	84.855	126.278
32	32.000	812.80	17	28.08	713.35	1.882	47.81	77.869	115.882
			21	28.83	732.29	1.524	38.70	63.835	94.997
			26	29.44	747.78	1.231	31.26	52.086	77.513
			32.5	29.95	760.78	0.985	25.01	42.023	62.538
			15.5	31.17	791.69	2.323	58.99	107.395	159.821
			17	31.60	802.52	2.118	53.79	98.553	146.663
36	36.000	914.40	21	32.43	823.83	1.714	43.54	80.791	120.231
			26	33.12	841.25	1.385	35.17	65.922	98.102
			32.5	33.70	855.88	1.108	28.14	53.186	79.149
			15.5	36.36	923.64	2.710	68.83	146.176	217.534
			17	36.86	936.27	2.471	62.75	134.141	199.625
42	42.000	1066.80	21	37.84	961.14	2.000	50.80	109.966	163.648
			26	38.64	981.46	1.615	41.03	89.727	133.528
			32.5	39.31	998.52	1.292	32.82	72.392	107.731

(See ASTM D3035, F714 and AWWA C-901/906 for OD and wall thickness tolerances).
(Weights are calculated in accordance with PPI TR-7).

Table A-2 (cont)
PIPE WEIGHTS AND DIMENSIONS *(IPS)*
PE3408 *(BLACK)*

OD			SDR	Nominal ID		Minimum Wall		Weight	
Nominal in.	Actual in.	Actual mm.		in.	mm.	in.	mm.	lb. per foot	kg. per meter
			7	11.25	285.64	2.286	58.06	42.786	63.673
			7.3	11.44	290.60	2.192	55.67	41.329	61.504
			9	12.30	312.48	1.778	45.16	34.598	51.487
			9.3	12.42	315.51	1.720	43.70	33.626	50.041
			11	12.97	329.55	1.455	36.95	28.994	43.149
16	16.000	406.40	11.5	13.11	332.89	1.391	35.34	27.862	41.464
			13.5	13.53	343.78	1.185	30.10	24.092	35.852
			15.5	13.85	351.86	1.032	26.22	21.214	31.570
			17	14.04	356.68	0.941	23.91	19.467	28.970
			21	14.42	366.15	0.762	19.35	15.959	23.749
			26	14.72	373.89	0.615	15.63	13.022	19.378
			7	12.65	321.35	2.571	65.31	54.151	80.586
			7.3	12.87	326.93	2.466	62.63	52.307	77.841
			9	13.84	351.54	2.000	50.80	43.788	65.164
			9.3	13.97	354.94	1.935	49.16	42.558	63.333
			11	14.60	370.75	1.636	41.56	36.696	54.610
18	18.000	457.20	11.5	14.74	374.51	1.565	39.76	35.263	52.478
			13.5	15.23	386.76	1.333	33.87	30.491	45.376
			15.5	15.58	395.85	1.161	29.50	26.849	39.955
			17	15.80	401.26	1.059	26.89	24.638	36.666
			21	16.22	411.92	0.857	21.77	20.198	30.058
			26	16.56	420.62	0.692	17.58	16.480	24.526
			32.5	16.85	427.94	0.554	14.07	13.296	19.787
			7	14.06	357.05	2.857	72.57	66.853	99.489
			7.3	14.30	363.25	2.740	69.59	64.576	96.100
			9	15.38	390.60	2.222	56.44	54.059	80.449
			9.3	15.53	394.38	2.151	54.62	52.541	78.189
			11	16.22	411.94	1.818	46.18	45.304	67.420
20	20.000	508.00	11.5	16.38	416.12	1.739	44.17	43.535	64.787
			13.5	16.92	429.73	1.481	37.63	37.643	56.019
			15.5	17.32	439.83	1.290	32.77	33.146	49.327
			17	17.55	445.84	1.176	29.88	30.418	45.266
			21	18.02	457.68	0.952	24.19	24.936	37.108
			26	18.40	467.36	0.769	19.54	20.346	30.279
			32.5	18.72	475.49	0.615	15.63	16.415	24.429
			9	16.92	429.66	2.444	62.09	65.412	97.343
			9.3	17.08	433.82	2.366	60.09	63.574	94.609
			11	17.84	453.14	2.000	50.80	54.818	81.578
			11.5	18.02	457.73	1.913	48.59	52.677	78.393
22	22.000	558.80	13.5	18.61	472.70	1.630	41.39	45.548	67.783
			15.5	19.05	483.81	1.419	36.05	40.107	59.686
			17	19.31	490.43	1.294	32.87	36.805	54.772
			21	19.82	503.45	1.048	26.61	30.172	44.901
			26	20.24	514.10	0.846	21.49	24.619	36.637
			32.5	20.59	523.04	0.677	17.19	19.863	29.559

(See ASTM D3035, F714 and AWWA C-901/906 for OD and wall thickness tolerances).
(Weights are calculated in accordance with PPI TR-7).

HYDROLOGIC EVALUATI[illegible]
LANDFILL PERFORMANCE (HE[illegible]DEL

The HELP Model was used to determine the leachate quantities for the leachate collection system as well as other useful information. The precipitation, evaporation, solar radiation, and temperature values that were used in the model were generated from default data corresponding to the Salt Lake area as designated in the HELP Model program. The climate data that was used correlated closely with average temperature and precipitation data reported in the Western Regional Climate Center database, found at www.wrcc.dri.edu. The locations used to compare were at Dugway and the Saltair Salt Plant. Some inputs for evapotranspiration and weather data were not covered in the default data. The evaporative zone depth was assumed to be 16 inches. The maximum leaf area index was assumed to be zero. These values were assumed based on the arid desert conditions that exist in this area.

The model was set up according to the preliminary designs for the layer system. From the HELP Model manual, Table 4 entitled "Default Soil, Waste, and Geosynthetic Characteristics" was used to determine which layer classification to use. The model used 6 - 9 layers depending on the phase of construction and are summarized below:

Layer	Thickness (in.)	Porosity (Vol/Vol)	Hydraulic Conductivity (cm/sec)
Erosion Protection Layer - Gravel	0 - 3	0.397	0.3
Soil Cover	0 - 24	0.473	5.2E-4
HDPE Liner	0 - 0.06	0.0	1.99E-13
Municipal Waste	0 - 2400	0.168	1.0E-3
Soil	24	0.473	5.2E-4
Geotextile	0.05	0.1	0.14
Drainage Net - Geonet	0.1	0.85	33.0
High Density Polyethylene - HDPE Liner	0.06	0.0	1.99E-13
GCL	0.25	0.75	4.99E-9

The HELP Model was run for different waste heights in order to determine the worst case condition. Once the full waste height was reached, the model was run with and without the closure cap. The results are summarized in the following table:

Model Run - Waste Height	Peak Daily Collected at Geonet (in.)	Annual Average Collected at Geonet (in.)	Annual Average Runoff (in.)
No Waste	0.13877	1.61251	0.071
10 Feet	0.21503	2.70216	0.069
50 Feet	0.20878	2.70228	0.069
100 Feet	0.24152	2.70227	0.069
200 Feet	0.22244	2.70228	0.069
Closure	0.00834	0.46316	0.142

No WASTE

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	12.69	(2.174)	921052.1	100.00
RUNOFF	0.071	(0.1112)	5135.54	0.558
EVAPOTRANSPIRATION	10.998	(1.8149)	798429.81	86.687
LATERAL DRAINAGE COLLECTED FROM LAYER 4	1.61251	(0.84207)	117068.195	12.71027
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.00000	(0.00000)	0.117	0.00001
AVERAGE HEAD ON TOP OF LAYER 5	0.001	(0.001)		
CHANGE IN WATER STORAGE	0.006	(0.7090)	418.28	0.045

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	113255.992
RUNOFF	0.259	18782.0410
DRAINAGE COLLECTED FROM LAYER 4	0.13877	10074.83890
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.000000	0.00154
AVERAGE HEAD ON TOP OF LAYER 5	0.035	
MAXIMUM HEAD ON TOP OF LAYER 5	0.071	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	1.06	77015.2031
MAXIMUM VEG. SOIL WATER (VOL/VOL)	0.1740	
MINIMUM VEG. SOIL WATER (VOL/VOL)	0.0402	

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	12.69	(2.174)	921052.1	100.00
RUNOFF	0.069	(0.1089)	5045.55	0.548
EVAPOTRANSPIRATION	9.918	(1.6315)	720081.19	78.180
LATERAL DRAINAGE COLLECTED FROM LAYER 4	2.70216	(0.94981)	196177.141	21.29925
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.00000	(0.00000)	0.170	0.00002
AVERAGE HEAD ON TOP OF LAYER 5	0.002	(0.001)		
CHANGE IN WATER STORAGE	-0.003	(0.5785)	-252.02	-0.027

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	113255.992
RUNOFF	0.258	18759.7109
DRAINAGE COLLECTED FROM LAYER 4	0.21503	15610.95510
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.000000	0.00206
AVERAGE HEAD ON TOP OF LAYER 5	0.055	
MAXIMUM HEAD ON TOP OF LAYER 5	0.106	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	27.2 FEET	
SNOW WATER	1.06	77015.2031
MAXIMUM VEG. SOIL WATER (VOL/VOL)		0.1328
MINIMUM VEG. SOIL WATER (VOL/VOL)		0.0190

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	12.69	(2.174)	921052.1	100.00
RUNOFF	0.069	(0.1089)	5045.55	0.548
EVAPOTRANSPIRATION	9.918	(1.6315)	720081.19	78.180
LATERAL DRAINAGE COLLECTED FROM LAYER 4	2.70227	(0.94762)	196184.625	21.30006
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.00000	(0.00000)	0.169	0.00002
AVERAGE HEAD ON TOP OF LAYER 5	0.002	(0.001)		
CHANGE IN WATER STORAGE	-0.004	(0.5801)	-259.50	-0.028

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	113255.992
RUNOFF	0.258	18759.7109
DRAINAGE COLLECTED FROM LAYER 4	0.20878	15157.25390
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.000000	0.00202
AVERAGE HEAD ON TOP OF LAYER 5	0.053	
MAXIMUM HEAD ON TOP OF LAYER 5	0.108	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	1.06	77015.2031
MAXIMUM VEG. SOIL WATER (VOL/VOL)		0.1328
MINIMUM VEG. SOIL WATER (VOL/VOL)		0.0190

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	12.69	(2.174)	921052.1	100.00
RUNOFF	0.069	(0.1089)	5045.55	0.548
EVAPOTRANSPIRATION	9.918	(1.6315)	720081.19	78.180
LATERAL DRAINAGE COLLECTED FROM LAYER 4	2.70228	(0.94740)	196185.625	21.30017
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.00000	(0.00000)	0.169	0.00002
AVERAGE HEAD ON TOP OF LAYER 5	0.002	(0.001)		
CHANGE IN WATER STORAGE	-0.004	(0.5803)	-260.52	-0.028

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	113255.992
RUNOFF	0.258	18759.7109
DRAINAGE COLLECTED FROM LAYER 4	0.24152	17534.43360
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.000000	0.00225
AVERAGE HEAD ON TOP OF LAYER 5	0.061	
MAXIMUM HEAD ON TOP OF LAYER 5	0.121	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	10.3 FEET	
SNOW WATER	1.06	77015.2031
MAXIMUM VEG. SOIL WATER (VOL/VOL)		0.1328
MINIMUM VEG. SOIL WATER (VOL/VOL)		0.0190

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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200'

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	12.69	(2.174)	921052.1	100.00
RUNOFF	0.069	(0.1089)	5045.55	0.548
EVAPOTRANSPIRATION	9.918	(1.6315)	720081.19	78.180
LATERAL DRAINAGE COLLECTED FROM LAYER 4	2.70228	(0.94730)	196185.641	21.30017
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.00000	(0.00000)	0.169	0.00002
AVERAGE HEAD ON TOP OF LAYER 5	0.002	(0.001)		
CHANGE IN WATER STORAGE	-0.004	(0.5804)	-260.50	-0.028

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	113255.992
RUNOFF	0.258	18759.7109
DRAINAGE COLLECTED FROM LAYER 4	0.22244	16149.48340
PERCOLATION/LEAKAGE THROUGH LAYER 6	0.000000	0.00211
AVERAGE HEAD ON TOP OF LAYER 5	0.057	
MAXIMUM HEAD ON TOP OF LAYER 5	0.109	
LOCATION OF MAXIMUM HEAD IN LAYER 4 (DISTANCE FROM DRAIN)	31.6 FEET	
SNOW WATER	1.06	77015.2031
MAXIMUM VEG. SOIL WATER (VOL/VOL)		0.1328
MINIMUM VEG. SOIL WATER (VOL/VOL)		0.0190

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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Closure

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AVERAGE ANNUAL TOTALS & (STD. DEVIATIONS) FOR YEARS 1 THROUGH 30

	INCHES		CU. FEET	PERCENT
PRECIPITATION	12.69	(2.174)	921052.1	100.00
RUNOFF	0.142	(0.1373)	10311.68	1.120
EVAPOTRANSPIRATION	12.058	(1.9901)	875443.69	95.048
LATERAL DRAINAGE COLLECTED FROM LAYER 2	0.01480	(0.01790)	1074.828	0.11670
PERCOLATION/LEAKAGE THROUGH LAYER 3	0.46317	(0.43227)	33626.137	3.65084
AVERAGE HEAD ON TOP OF LAYER 3	1.130	(1.368)		
LATERAL DRAINAGE COLLECTED FROM LAYER 7	0.46316	(0.44777)	33625.562	3.65078
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.00000	(0.00000)	0.012	0.00000
AVERAGE HEAD ON TOP OF LAYER 8	0.000	(0.000)		
CHANGE IN WATER STORAGE	0.008	(0.9827)	596.26	0.065

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PEAK DAILY VALUES FOR YEARS 1 THROUGH 30

	(INCHES)	(CU. FT.)
PRECIPITATION	1.56	113255.992
RUNOFF	0.344	24941.7246
DRAINAGE COLLECTED FROM LAYER 2	0.00038	27.51882
PERCOLATION/LEAKAGE THROUGH LAYER 3	0.008502	617.27661
AVERAGE HEAD ON TOP OF LAYER 3	10.570	
MAXIMUM HEAD ON TOP OF LAYER 3	20.450	
LOCATION OF MAXIMUM HEAD IN LAYER 2 (DISTANCE FROM DRAIN)	123.9 FEET	
DRAINAGE COLLECTED FROM LAYER 7	0.00834	605.28369
PERCOLATION/LEAKAGE THROUGH LAYER 9	0.000000	0.00006
AVERAGE HEAD ON TOP OF LAYER 8	0.001	
MAXIMUM HEAD ON TOP OF LAYER 8	0.001	
LOCATION OF MAXIMUM HEAD IN LAYER 7 (DISTANCE FROM DRAIN)	0.0 FEET	
SNOW WATER	1.06	77015.2031
MAXIMUM VEG. SOIL WATER (VOL/VOL)		0.2673
MINIMUM VEG. SOIL WATER (VOL/VOL)		0.0869

*** Maximum heads are computed using McEnroe's equations. ***

Reference: Maximum Saturated Depth over Landfill Liner
by Bruce M. McEnroe, University of Kansas
ASCE Journal of Environmental Engineering
Vol. 119, No. 2, March 1993, pp. 262-270.

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TECHNICAL NOTE ON USING HELP MODEL (VER. 3.07)

I: INPUT STEPS GUIDE

The purpose of this document is to help the users of HELP Model through the input procedures, and interpretation of the output results . All information contained herein are from HELP "User's Guide" and "Engineering Documentation" for version 3. Included is a step-by-step example, which is a part of the GRI report # 19, page 34- 37 (leachate collection system).

INSTALLATION NOTE

You can download the latest version of HELP Model 3.07 from the following web-site address: http://www.wes.army.mil/el/elmodels/index.html. You will save the downloaded file (zhelp3w.exe or zhelp3p.exe) onto a temporary subdirectory, after you execute the file it will be self extracted into some files needed for the setup. From the files that have been self extracted, you run the setup file follow the steps that will show on the screen.

Whether you download HELP Model program from the internet or install it from a floppy, the files should be installed (or copied) in a subdirectory directly under the root, i.e. C:\ or D:\. The executable file is called "Help3.bat".

INPUT STEPS

1. Weather Data

From the main menu you choose option 1 **"Enter/ Edit Weather Data"**, this will prompt you to another screen with the following four options:

Evapotranspiration; Precipitation; Temperature; and Solar radiation

For each you hit "PgDn" to start new file, or "F4" to choose from a list of saved files. Below is a description of the input data required for each of the four weather selections.

1.1 Evapotranspiration

Evapotranspiration is the first weather option the program is going to prompt you for if you are starting a new project. However, if you're editing an existing project you'll be prompted to the screen corresponding to your selection of either of the four weather data.

a) Units: with up or down arrows you select either 1 Customary (English), or 2 Metric. In the current example we selected **Metric.**

b) City: If you're going to select **default** option, you hit "F5" to select a "State" first and then a "City" that is closest to the landfill location, then all the corresponding required data will be filled except for the following two data:

"Evaporative Zone Depth" in centimeters which is at least equal to the expected average depth of root penetration. To the right of the screen a table with three columns will appear that indicates the input value, you choose a value depending on the condition of vegetation expected.

Bare	Fair	Excellent
25	55	101

In our example we'll select **Texas, Austin, 25 cm** (for no vegetation)

"Leaf Area Index" (LAI), LAI is a dimensionless ratio of the leaf area that is actively transpiring vegetation to the nominal surface area of the land on which the vegetation is growing. Below is a table that lists the LAI values for different conditions of vegetation.

Bare	Poor Stand of Grass	Fair Stand of Grass	Good Stand of Grass	Excellent Stand of Grass
0.0	1.0	2.0	3.5	5.0

In our example we'll choose **0.0** for no vegetation condition.

If you're going to select the **manual** option, in addition to the above two parameters you'll be asked to input the following parameters: location (city and state), dates of starting and ending the growing season, normal average annual wind speed, and Normal average quarterly relative humidity. The last three data are available from "Climatic Atlas of the United States" (NOAA, 1974)

1.2 Precipitation, Temperature, Solar Radiation

The input options for the above three weather data are: **Synthetic, Create/Edit, NOAA tape, Climatedata, ASCII file, HELP Version 2, and Canadian Climatological**. Only Precipitation has an extra option which is **Default**. Below is a description of the input options:

Default (Precipitation only): The user may select any of the stored 102 cities for which the historical precipitation data are recorded during 5 years from 1974 to 1978. In the current example this option is chosen and the city of **San Antonio, Texas** is selected.

Synthetic: the program will generate from 1 to 100 years of daily Precipitation, Temperature, or Solar Radiation data stochastically for the selected location using a synthetic weather generator. The user may enter normal mean monthly precipitation values for the location to improve the statistical characteristics of the resulted daily values. For that option user needs to specify a location from 139 stored cities, number of years of data to be generated, and normal mean monthly value (optional).
For the current example the synthetic option is chosen for both temperature and solar radiation data where the city of **Austin, Texas**, and **5** years are selected.

Create/Edit, NOAA tape, Climatedata, ASCII file, HELP Version 2, and Canadian Climatological: all of these 6 options require the user to input the location (city and state), and the corresponding daily precipitation, temperature, or solar radiation data stored in a saved file(s) name, the format of the file(s) differs from option to the other. All options accept customary or metric units.

After completing entering the weather data input, you hit "F10" to end and save by typing the path and the name of each of the four saved files. The files will take automatically a default extensions as: D4; D7; D13; and D11 for Precipitation; Temperature; Solar radiation; and Evapotranspiration respectively (do not attempt to change the default extensions). After saving the files, you'll be prompted to the main menu screen. The program will prompt you to a warning screen if one or more of the data is missed or incorrect.

2. Soil Data

From the main menu you choose option 2 **"Enter/ Edit Soil Data"**, this will prompt you to another screen where you either hit "PgDn" to start new file, or "F4" to choose from a list of saved files. Below is a description of the input soil data:

2.1 Initial Information

The first screen of soil data input contains the following required information:

Unit System: on the same screen you are prompted to select a unit system, in the current example we selected **Metric**, Then you're prompted to another screen where you input;

Project Title: in the current example: **"Example in GRI Report # 19"**

Landfill Area: in the current example: **4 hectares**

Percent of landfill where runoff is possible: in the current example **100%**

Method of initialization of moisture storage: you have two options: 1) to choose to enter the initial moisture content for the soil layers in the analyzed profile as per the available soil information, and then at the following screen you'll input the corresponding values. 2) to let the program initialize the moisture content to the near steady-state condition, **option (2)** is selected in the current example.

Initial Snow/Water Storage: this piece of information is optional and needed when moisture storage is user-defined.

2.2 Layers Information

The second, third, and fourth screens contain the layers information as follows:

2.2.1 General Soil Information

Layer Type: four types of layers are supported by HELP model; 1)vertical percolation, 2) lateral drainage, 3) barrier soil liner, and 4) geomembrane liner

Layer thickness: in customary or Metric systems

Soil Texture: the soil texture information contains four properties;

- Porosity (vol/vol)
- Field Capacity (vol/vol)
- Witlting point (vol/vol), and
- Saturated hydraulic conductivity (cm/sec)

The user has the option to select from a 42 default soil/ material textures, select from user-built soil texture library where the properties will be automatically assigned, or to enter the above information manually. To learn more about the above properties refer to section "3.5 Soil Characteristics" of HELP Model User's Guide.

Initial moisture storage: vol/vol, optional if you choose option (1) of "Method of initialization of moisture storage" in section 2.1.

Rate of subsurface inflow to layer: optional, customary or Metric unit systems (mm or inch/year).

2.2.2 Layer Specific Information

The four types of layers that are supported by HELP model are explained below:

Vertical Percolation Layer: waste and vegetation support layers are examples of vertical percolation layer. The downward flow in the vertical percolation layer is modeled by the unsaturated vertical gravity drainage. The upward flux due to evapotranspiration is modeled as an extraction.

Lateral Drainage Layer: the lateral drainage layer is designed to promote drainage laterally to a collection and removal system. The vertical flow in this layer is modeled as in the vertical percolation layer, however, a saturated lateral drainage is also allowed. In addition to the soil data in section 2.2.1, the following information are also required to model the lateral drainage layer:

- Max drainage length: customary or Metric. The horizontal projection of the slope, rather than the distance along the slope.
- Drain slope: percent. From 0 to 50 percent

- Percentage of recirculated to collected leachate. From 0 to 100%
- Layer No. to receive the recirculated leachate. Vertical percolation or lateral drainage. Layer number.

Barrier soil liners: are intended to restrict vertical drainage/ leakage/ percolation. These layers should have significantly lower hydraulic conductivity than the other layers. The barrier soil layer is assumed to be saturated all time but leak only when there is a positive head on the top surface of the liner. HELP model allows only downward saturated flow through the barrier soil layer, thus any water moving into the liner will eventually percolate through it. Evapotranspiration and lateral drainage are not permitted.

Geomembrane liners: are virtually impermeable synthetic membranes that reduce the area of vertical drainage/ leakage/ percolation to a very small fraction of the area locatednear manufacturing flaws and installation defects. Also a small quantity of vapor transport is modeled by specifying the vapor diffusivity of the geomembrane liner. In addition to data listed in section 2.2.1, the following information is required:

- Pinhole density: (#/acre or hectare). Defects of a diameter equal or smaller than the membrane thickness (estimated as 1 mm in diameter). Typical geomembranes may have from 0.5 to 1 pinhole per acre (1 to 2 per hectare).

- Installation defects density: (#/acre or hectare). Defects of a diameter greater than the membrane thickness (estimated as 1 cm^2 in area).

Installation Quality	Defect Density (#/acre)	Frequency (%)
Excellent	Up to 1	10
Good	1 to 4	40
Fair	4 to 10	40
Poor	10 to 20 (old landfills)	10

- Placement quality: addresses the quality of contact between geomembrane and the underneath soil that limits the drainage rate. The table below explains the 6 cases supported by HELP model:

1. Perfect	Assumes perfect, (no gap, "sprayed-on" seal)
2. Excellent	Assumes exceptional contact (typically achievable only in the lab)
3. Good	Assumes good field installation with well-prepared, smooth soil surface and geomembrane wrinkle control
4. Poor	Assumes poor field installation with a less well-prepared soil surface and/ or geomembrane wrinkling control
5. Worst Case	Assumes that contact between geomembrane and the underneath does not limit drainage rate

6. Separating Geotextile Assumes leakage spreading and rate is controlled by the in-plane transmissivity of the geotextile separating the geomemebrane and the adjacent soil layer. This quality does not apply to GCL where bentonite swells upon wetting and extrudes into the geotextile significantly reducing its ability to spread the leakage.

- Saturated hydraulic conductivity: (vapor diffusivity), cm/sec
- Geotextile in-plane transmissivity, cm^2/sec (optional when placed with geomembrane)

In the current example two layers are simulated, the following is the information required from the user as input. Other information is set up as default values corresponding to the layer's texture number:

1) Lateral drainage layer

   - Type 2
   - thickness 45 cm
   - texture number 21
   - slope length 10 m
   - slope: 33%
   - percent of recirculated leachate; zero%

2) Geomembrane liner

   - Type 4
   - thickness 0.15 cm
   - texture number 35
   - zero pinholes and zero installation defects
   - placement quality: 1 (perfect)

2.3 Site Characteristics

The third screen contains the runoff curve number information, the user has three options to input the SCS runoff curve number: 1) defined by the user, 2) defined by the user and modified

by HELP model for slope surface and length, and 3) computed by HELP model based on top layer texture, slope length and slope.
In the current example **option 3** is selected and the corresponding slope %, slope length, soil texture and vegetation conditions (1: bare, 2:poor, 3: fair, 4:good, 5: excellent stand of grass) are input as in the previous step for the top layer (drainage layer). The SCS runoff curve number calculated by HELP model is **75.9**.

After completing entering the soil data input, you hit "F10" to end and save by typing the path and name of the file, the file will take automatically a default extension as: D10 (do not attempt to change the default extensions). After saving the file, you'll be prompted to the main menu screen. The program will prompt you to a warning screen if one or more of the data is missed or incorrect.

3. Execution, Viewing and Printing Results

From the main menu you choose option 3 "**Execute Simulation**" which will prompt you to a screen where you type the five files' names which contain weather and soil data information. Then to another screen where the program asks for the unit system wanted for the output (regardless of the system used in the input data), number of years during which the output is generated, and the intervals of the generated output; annual, monthly, or daily. The program will take few minuets (variable depending on your computer speed) to execute the project information, then it'll prompt to the main menu. To view or to print* the out put you choose either option 4 "**View Results**", or option 5 "**Print Results**".

A printout of the example discussed above is included.

*Since HELP model is DOS operated program, a conflict in the printing command may occur. It's recommended to open and print the output file "**filename.out**" through the program "**Notepad**" found in your Windows 95 system under: "start/programs/accessories/notepad".

4. Flux Calculations

Referring to the output table: "Peak Daily Values for Years 1974 Through 1978", drainage collected from layer 1= 61.12513 mm (0.061 m/day)

Hourly Flux (m^3/hr)/ width (m) = Depth of Liquid Collected Daily (m/day) x Slope length (m) / 24 (hr/day)
=(0.061)*(10)/24 = 0.025 m^3/hr-m width

II: DRAINAGE GEOCOMPOSITE INPUT DATA

As discussed in section I, the input data for the lateral drainage Layer (Layer Type 2) could be divided into two categories; 1) project specific , and 2) product specific. The properties under the project specific category are listed on page 4 of section I. This section discusses the product specific properties for the lateral drainage layer with an emphasis on geosynthetic drainage geocomposites. In general, it should be noted that unlike the conventional soil drainage layer (sand or aggregate), the physical and hydraulic properties of geosynthetic materials are highly dependent on project's design criteria, such as anticipated normal load, hydraulic gradient, and boundary conditions. The five required properties for the drainage layer are as follows:

1. Thickness (mm, inch)
The layer thickness determined at the anticipated normal load.

2. Porosity (vol/vol)
The volume of space/total volume.

3. Field Capacity (vol/vol)
Field capacity as defined in HELP Model is the amount of water that the product will accept before gravity flow could commence in the layer.

4. Wilting Point (vol/vol)
Wilting point by definition is the maximum amount of moisture in the material that can not be drawn by plants

5. Saturated Hydraulic Conductivity (cm/sec)
The saturated hydraulic conductivity of the geonets are determined by dividing the transmissivity measured under the required design and field conditions by the corresponding thickness of the geonet.

The table below presents the above discussed properties for two of Tenax's geocomposites; Tenflow and Tendrain used typically for landfill capping and lining applications respectively.

Tenax's Lateral Drainage Layer Input Data for HELP Model

Geonet Type	Thickness* (mm/mils)	Porosity (vol/vol)	Field Capacity+ (vol/vol)	Wilting Point+ (vol/vol)	Saturated Hydraulic Conductivity++ (cm/sec)
Tenflow	7.30/ 287	0.86	0.01	0.005	15.8
Tendrain	5.14/ 202	0.70	0.01	0.005	12.4

*Measured at anticipated stress level of 1,000 psf for Tenflow, and 15,000 for Tendrain (geonet only)
+ Per HELP Model default value for drainage geonets
++Determining the Design Hydraulic Conductivity for Drainage Geocomposites.

Equations:

$$T_{all} = \frac{T_{ult}}{RFin*RFcr*RFcc*RFbc} \qquad (1)$$

Where, T_{all} = allowable Transmissivity [cm^2/s]

T_{ult} = ultimate Transmissivity measured in the lab [cm^2/s]

RF_{in} = reduction factor for intrusion of adjacent geotextile

RF_{cr} = reduction factor for creep deformation

RF_{cc} = reduction factor for chemical clogging

RF_{bc} = reduction factor for biological clogging

$$T_{dsg} = \frac{T_{all}}{FS} \qquad (2)$$

Where, T_{dsg} = design Transmissivity used in calculations [cm^2/s]

FS = overall factor of safety

$$T_{dsg} = k_{dsg}*t_{dsg} \qquad (3)$$

Where, k_{dsg} = design hydraulic conductivity used in calculations [cm^2/s]

t_{dsg} = design thickness used in calculations [cm]

Solution:

Landfill Final Closure:

1) Estimated design load on landfill foundation = 1,000 psf

2) Ultimate Transmissivity = T_{ult} = 4.0 * 10E-3 m^2/sec = 40 cm^2/s

(geocomposite tested in soil boundary condition under 1,000 psf, a hydraulic gradient of 0.33, and a seating period of 100 hours)

3) Using Table 1 for typical values of reduction factors, *Giroud, Zornberg, and Zhao, 2000, "Hydraulic Design of Liquid Collection Layers", Geosynthetics International*: RFin = 1.1, RFcc = 1.1, RFbc = 1.4

4) Using RFcr = 1.02 (determined value for Tenflow)

5) FS = 2.0 (state of practice typical value)

6) **t_{dsg} = 0.730 cm (0.287 inches)**

Substituting in Equation (1): T_{all} = 23.1 cm^2/sec

Substituting in Equation (2): T_{dsg} = 11.6 cm^2/sec

Substituting in Equation (3): **k_{dsg} = 15.8 cm/sec**

Landfill Liner Prior to Final Closure:

1) Estimated design load on landfill foundation = 15,000 psf

2) Ultimate Transmissivity = T_{ult} = 5.0 * 10E-3 m^2/sec = 50 cm^2/s

(geocomposite tested in soil boundary condition under 15,000 psf, a hydraulic gradient of 0.02, and a seating period of 100 hours)

3) Using Table 1 for typical values of reduction factors, *Giroud, Zornberg, and Zhao, 2000, "Hydraulic Design of Liquid Collection Layers", Geosynthetics International*: RFin = 1.2, RFcc = 1.75, RFbc = 1.75

4) Using RFcr = 1.07 (determined value for Tendrain)

5) FS = 2.0 (state of practice typical value)

6) **t_{dsg} = 0.514 cm (0.202 inches)**

Substituting in Equation (1): T_{all} = 12.7 cm^2/sec

Substituting in Equation (2): T_{dsg} = 6.4 cm^2/sec

Substituting in Equation (3): **k_{dsg} = 12.4 cm/sec**

Please note that the above calculations were done assuming typical information for the design requirements of a landfill liner and a landfill cap systems, as well as product design data for specific drainage geocomposites. The design engineer should implement the design data that are representative to the project in design and the considered products.

TABLE 4. DEFAULT SOIL, WASTE, AND GEOSYNTHETIC CHARACTERISTICS

Classification			Total Porosity	Field Capacity	Wilting Point	Saturated Hydraulic Conductivity	
HELP	USDA	USCS	vol/vol	vol/vol	vol/vol	cm/sec	
1	CoS	SP	0.417	0.045	0.018	1.0×10^{-2}	
2	S	SW	0.437	0.062	0.024	5.8×10^{-3}	
3	FS	SW	0.457	0.083	0.033	3.1×10^{-3}	
4	LS	SM	0.437	0.105	0.047	1.7×10^{-3}	
5	LFS	SM	0.457	0.131	0.058	1.0×10^{-3}	
6	SL	SM	0.453	0.190	0.085	7.2×10^{-4}	
7	FSL	SM	0.473	0.222	0.104	5.2×10^{-4}	
8	L	ML	0.463	0.232	0.116	3.7×10^{-4}	
9	SiL	ML	0.501	0.284	0.135	1.9×10^{-4}	
10	SCL	SC	0.398	0.244	0.136	1.2×10^{-4}	
11	CL	CL	0.464	0.310	0.187	6.4×10^{-5}	
12	SiCL	CL	0.471	0.342	0.210	4.2×10^{-5}	
13	SC	SC	0.430	0.321	0.221	3.3×10^{-5}	
14	SiC	CH	0.479	0.371	0.251	2.5×10^{-5}	
15	C	CH	0.475	0.378	0.265	1.7×10^{-5}	
16	Barrier Soil		0.427	0.418	0.367	1.0×10^{-7}	
17	Bentonite Mat (0.6 cm)		0.750	0.747	0.400	3.0×10^{-9}	GCL 1e 5×10^{-9}
18	Municipal Waste (900 lb/yd³ or 312 kg/m³)		0.671	0.292	0.077	1.0×10^{-3}	Waste
19	Municipal Waste (channeling and dead zones)		0.168	0.073	0.019	1.0×10^{-3}	
20	Drainage Net (0.5 cm)		0.850	0.010	0.005	$1.0\times10^{+1}$	Geonet
21	Gravel		0.397	0.032	0.013	3.0×10^{-1}	
22	L*	ML	0.419	0.307	0.180	1.9×10^{-5}	
23	SiL*	ML	0.461	0.360	0.203	9.0×10^{-6}	
24	SCL*	SC	0.365	0.305	0.202	2.7×10^{-6}	
25	CL*	CL	0.437	0.373	0.266	3.6×10^{-6}	
26	SiCL*	CL	0.445	0.393	0.277	1.9×10^{-6}	
27	SC*	SC	0.400	0.366	0.288	7.8×10^{-7}	
28	SiC*	CH	0.452	0.411	0.311	1.2×10^{-6}	
29	C*	CH	0.451	0.419	0.332	6.8×10^{-7}	
30	Coal-Burning Electric Plant Fly Ash*		0.541	0.187	0.047	5.0×10^{-5}	
31	Coal-Burning Electric Plant Bottom Ash*		0.578	0.076	0.025	4.1×10^{-3}	
32	Municipal Incinerator Fly Ash*		0.450	0.116	0.049	1.0×10^{-2}	
33	Fine Copper Slag*		0.375	0.055	0.020	4.1×10^{-2}	
34	Drainage Net (0.6 cm)		0.850	0.010	0.005	3.3×10^{-1}	Geonet

* Moderately Compacted .635 cm (Continued)

TABLE 4 (continued). DEFAULT SOIL, WASTE, AND GEOSYNTHETIC CHARACTERISTICS

Classification		Total Porosity	Field Capacity	Wilting Point	Saturated Hydraulic Conductivity
HELP	Geomembrane Material	vol/vol	vol/vol	vol/vol	cm/sec
35	High Density Polyethylene (HDPE)				$2.0x10^{-13}$
36	Low Density Polyethylene (LDPE)				$4.0x10^{-13}$
37	Polyvinyl Chloride (PVC)				$2.0x10^{-11}$
38	Butyl Rubber				$1.0x10^{-12}$
39	Chlorinated Polyethylene (CPE)				$4.0x10^{-12}$
40	Hypalon or Chlorosulfonated Polyethylene (CSPE)				$3.0x10^{-12}$
41	Ethylene-Propylene Diene Monomer (EPDM)				$2.0x10^{-12}$
42	Neoprene				$3.0x10^{-12}$

Membrane

(concluded)

user-defined soil option accepts non-default soil characteristics for layers assigned soil type numbers greater than 42. This is especially convenient for specifying characteristics of waste layers. User-specified soil characteristics can be assigned any soil type number greater than 42.

When a default soil type is used to describe the top soil layer, the program adjusts the saturated hydraulic conductivities of the soils in the top half of the evaporative zone for the effects of root channels. The saturated hydraulic conductivity value is multiplied by an empirical factor that is computed as a function of the user-specified maximum leaf area index. Example values of this factor are 1.0 for a maximum LAI of 0 (bare ground), 1.8 for a maximum LAI of 1 (poor stand of grass), 3.0 for a maximum LAI of 2 (fair stand of grass), 4.2 for a maximum LAI of 3.3 (good stand of grass) and 5.0 for a maximum LAI of 5 (excellent stand of grass).

The manual option requires values for porosity, field capacity, wilting point, and saturated hydraulic conductivity. These and related soil properties are defined below.

Soil Water Storage (Volumetric Content): the ratio of the volume of water in a soil to the total volume occupied by the soil, water and voids.

Total Porosity: the soil water storage/volumetric content at saturation (fraction of total volume).

POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

Utah 40.84902°N 112.75142°W 4271 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 3
G.M. Bonnin, D. Todd, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2003

Extracted: Mon Aug 9 2004

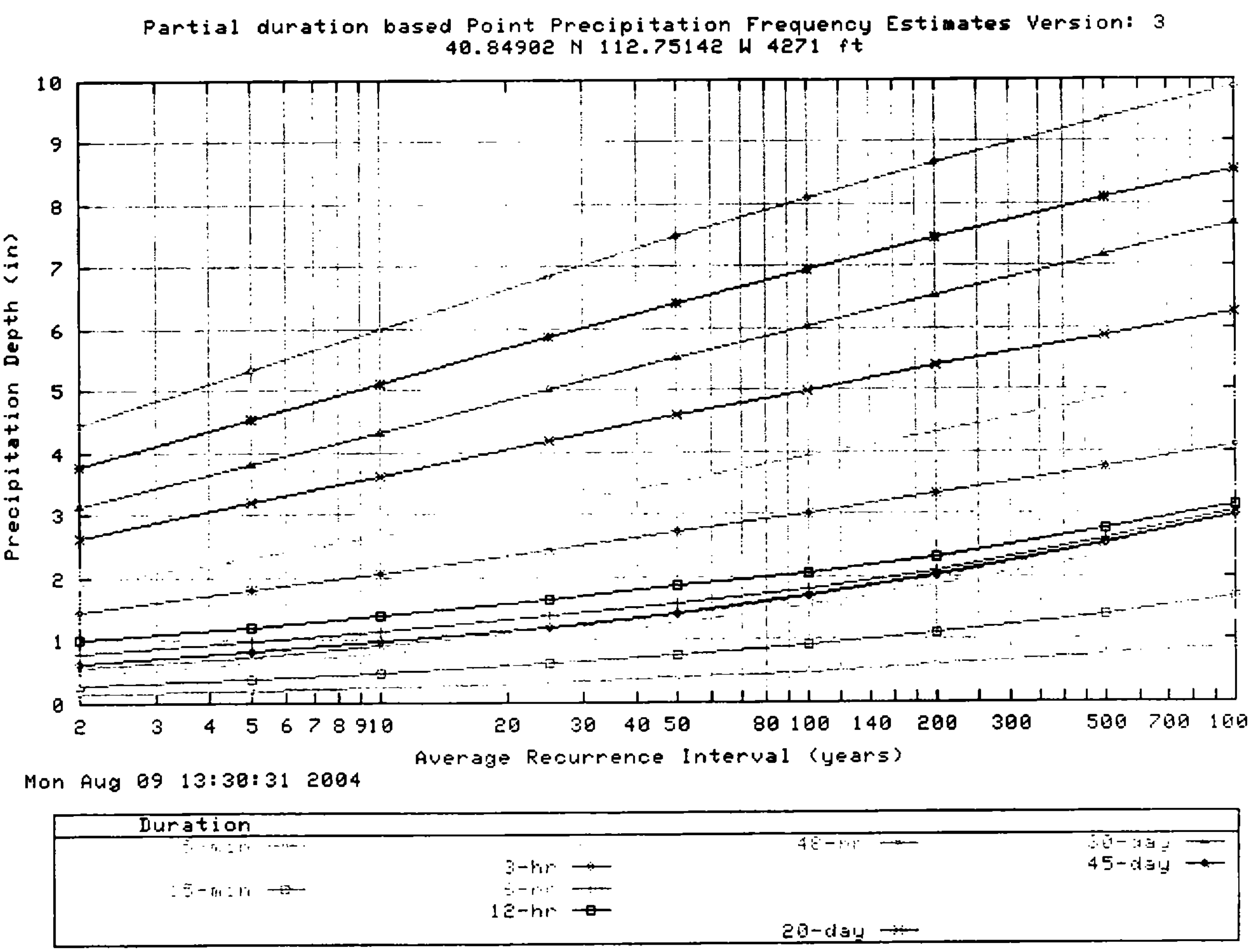
Confidence Limits | Seasonality | Location Maps | Other Info. | Grids | Maps | Help | Docs | U.S. M

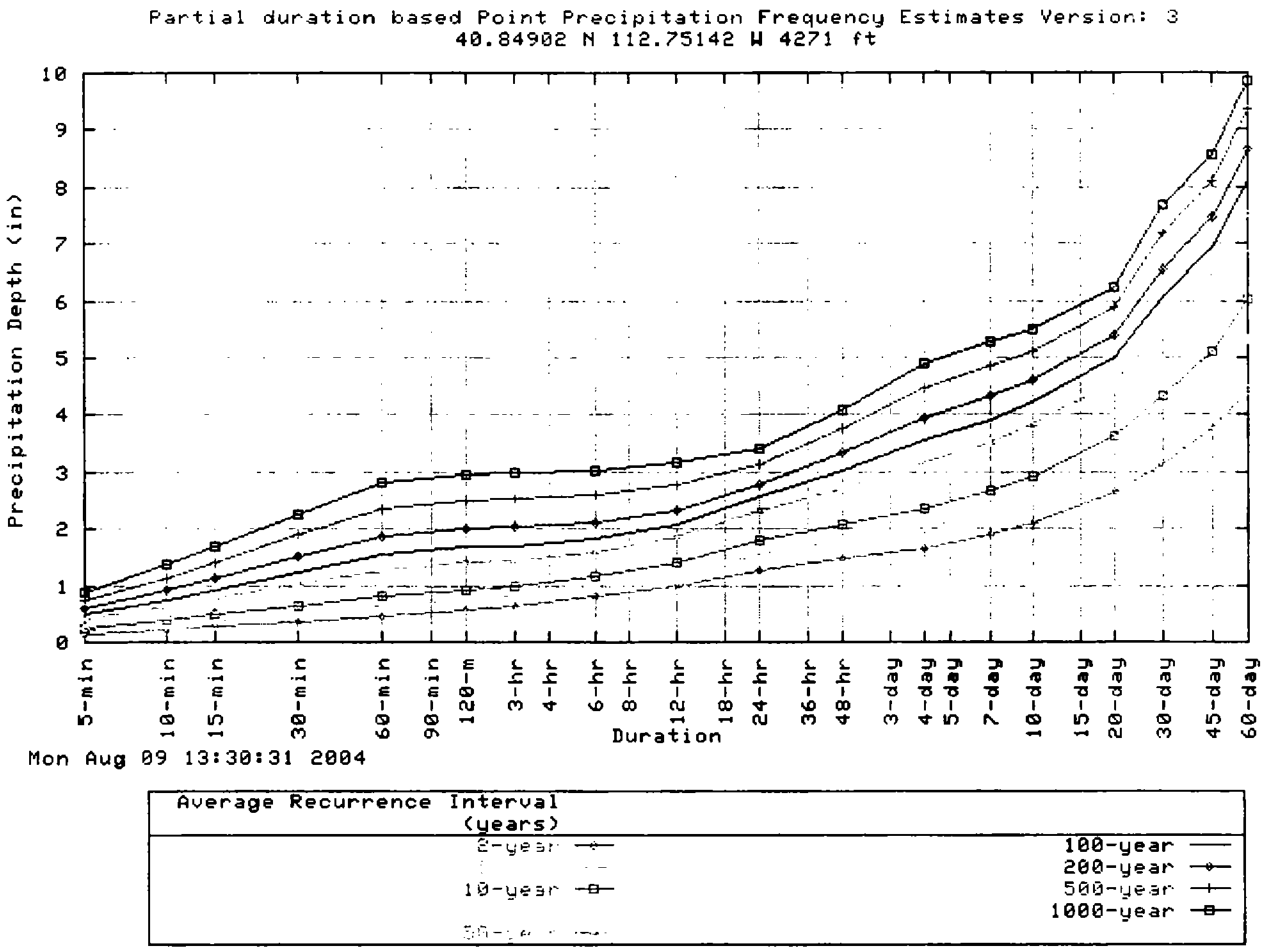
Precipitation Frequency Estimates (inches)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	0.14	0.22	0.27	0.37	0.46	0.56	0.64	0.81	1.00	1.28	1.47	1.66	1.89	2.10	2.64	3.13	3.77	4.44
5	0.20	0.31	0.38	0.51	0.63	0.74	0.82	0.99	1.22	1.56	1.80	2.05	2.32	2.56	3.20	3.80	4.53	5.33
10	0.25	0.38	0.48	0.64	0.79	0.91	0.97	1.15	1.40	1.78	2.06	2.37	2.68	2.94	3.63	4.32	5.12	6.01
25	0.33	0.51	0.63	0.85	1.05	1.17	1.22	1.39	1.66	2.09	2.44	2.82	3.16	3.44	4.18	5.01	5.86	6.87
50	0.41	0.62	0.77	1.03	1.28	1.41	1.44	1.58	1.86	2.32	2.73	3.17	3.54	3.82	4.59	5.53	6.41	7.49
100	0.49	0.75	0.93	1.25	1.55	1.68	1.71	1.81	2.07	2.56	3.03	3.55	3.92	4.21	5.00	6.04	6.95	8.09
200	0.59	0.90	1.12	1.50	1.86	2.00	2.02	2.11	2.32	2.79	3.34	3.93	4.32	4.60	5.39	6.54	7.45	8.66
500	0.75	1.14	1.41	1.90	2.35	2.51	2.53	2.60	2.78	3.12	3.76	4.47	4.86	5.12	5.89	7.19	8.09	9.37
1000	0.89	1.36	1.68	2.26	2.80	2.96	2.98	3.04	3.15	3.40	4.09	4.89	5.28	5.51	6.25	7.67	8.55	9.88

Text version of table

* These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence Interval. Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.





Confidence Limits -

* Upper bound of the 90% confidence interval Precipitation Frequency Estimates (inches)																		
ARI (years)**	**5 min**	**10 min**	**15 min**	**30 min**	**60 min**	**120 min**	**3 hr**	**6 hr**	**12 hr**	**24 hr**	**48 hr**	**4 day**	**7 day**	**10 day**	**20 day**	**30 day**	**45 day**	**60 day**
2	0.17	0.26	0.32	0.42	0.53	0.64	0.71	0.88	1.09	1.42	1.63	1.84	2.10	2.33	2.91	3.46	4.15	4.90
5	0.23	0.35	0.44	0.59	0.73	0.84	0.91	1.08	1.32	1.74	2.00	2.27	2.58	2.85	3.53	4.21	5.00	5.89
10	0.29	0.44	0.55	0.74	0.91	1.02	1.08	1.25	1.52	1.98	2.29	2.62	2.97	3.26	4.00	4.79	5.63	6.63
25	0.38	0.58	0.72	0.98	1.21	1.32	1.37	1.52	1.80	2.32	2.71	3.12	3.51	3.82	4.62	5.55	6.46	7.58
50	0.47	0.72	0.89	1.20	1.48	1.60	1.64	1.76	2.04	2.58	3.04	3.51	3.92	4.25	5.07	6.12	7.07	8.28
100	0.58	0.88	1.09	1.47	1.82	1.94	1.98	2.07	2.32	2.85	3.38	3.93	4.36	4.69	5.53	6.70	7.66	8.96
200	0.71	1.07	1.33	1.79	2.22	2.35	2.39	2.47	2.64	3.12	3.74	4.38	4.81	5.14	5.97	7.27	8.24	9.61
500	0.91	1.39	1.72	2.32	2.87	3.02	3.06	3.13	3.22	3.51	4.23	5.00	5.45	5.76	6.55	8.05	8.97	10.45
1000	1.11	1.68	2.09	2.81	3.48	3.63	3.68	3.75	3.80	3.85	4.62	5.50	5.94	6.22	6.99	8.62	9.52	11.04

* The upper bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are greater than.
** These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence Interval.
Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

* Lower bound of the 90% confidence interval Precipitation Frequency Estimates (inches)																		
ARI (years)**	**5 min**	**10 min**	**15 min**	**30 min**	**60 min**	**120 min**	**3 hr**	**6 hr**	**12 hr**	**24 hr**	**48 hr**	**4 day**	**7 day**	**10 day**	**20 day**	**30 day**	**45 day**	**60 day**

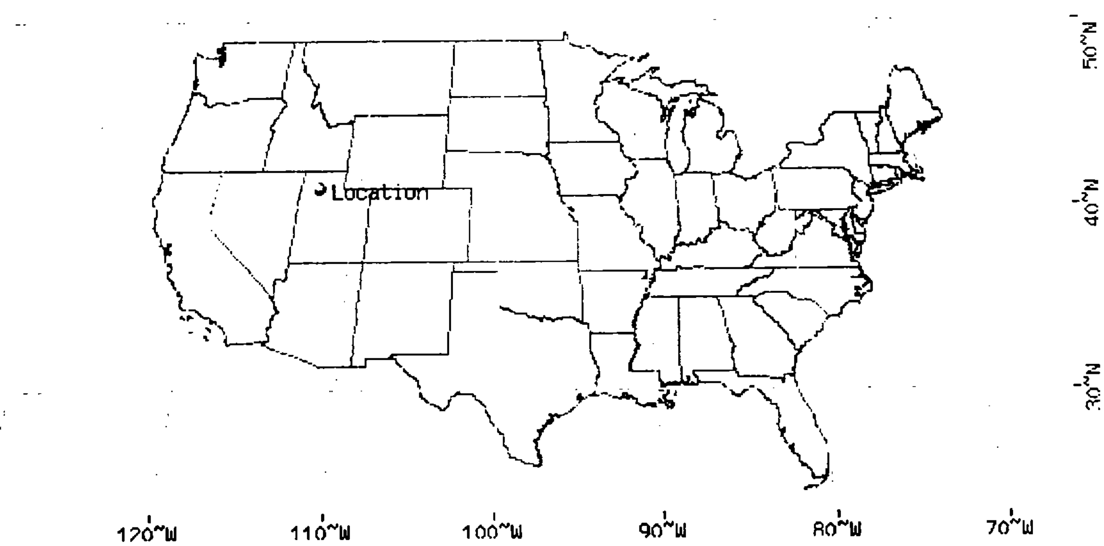
2	0.13	0.19	0.24	0.32	0.40	0.51	0.58	0.75	0.93	1.16	1.33	1.51	1.72	1.90	2.39	2.83	3.44	4.02
5	0.18	0.27	0.33	0.45	0.55	0.67	0.74	0.91	1.13	1.41	1.63	1.86	2.10	2.31	2.90	3.45	4.13	4.83
10	0.22	0.33	0.41	0.56	0.69	0.80	0.88	1.06	1.29	1.61	1.86	2.15	2.42	2.65	3.29	3.91	4.65	5.43
25	0.28	0.43	0.53	0.72	0.89	1.01	1.08	1.26	1.51	1.88	2.19	2.54	2.85	3.08	3.78	4.51	5.32	6.19
50	0.34	0.51	0.63	0.85	1.06	1.19	1.24	1.42	1.68	2.07	2.44	2.85	3.16	3.42	4.13	4.96	5.80	6.73
100	0.40	0.60	0.75	1.01	1.25	1.38	1.44	1.59	1.85	2.27	2.69	3.15	3.49	3.74	4.49	5.40	6.26	7.24
200	0.46	0.70	0.87	1.17	1.45	1.59	1.67	1.81	2.04	2.46	2.95	3.47	3.82	4.07	4.81	5.82	6.69	7.73
500	0.56	0.85	1.05	1.41	1.75	1.90	1.99	2.17	2.37	2.72	3.28	3.89	4.25	4.47	5.23	6.34	7.21	8.31
1000	0.64	0.97	1.20	1.61	2.00	2.15	2.27	2.48	2.64	2.94	3.53	4.21	4.57	4.78	5.52	6.72	7.58	8.71

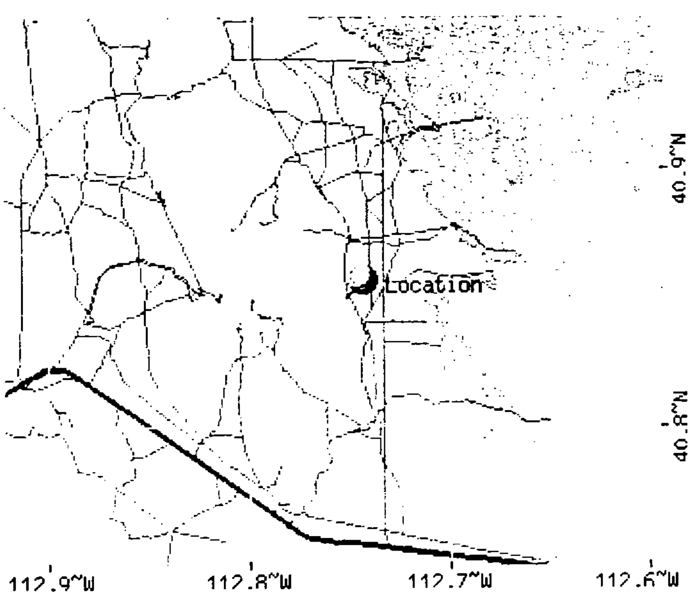
* The lower bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are less than.

** These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence Interval.

Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

Maps -





These maps were produced using a direct map request from the U.S. Census Bureau Mapping and Cartographic Resources Tiger Map Server.

Please read disclaimer for more information.

LEGEND

State
County
Indian Resv
Lake/Pond/Ocean
Street
Expressway
Highway
Connector
Stream
Military Area
National Park
Other Park
City
County

Scale 1:228583 0 2 4 6 8 mi / 0 2 4 6 8 10 km
*average--true scale depends on monitor resolution

Other Maps/Photographs -

View USGS Digital Raster Graphic (DRG) covering this location from TerraServer; **USGS Aerial Photograph** may also be available from this site. A DRG is a digitized version of a USGS topographic map. Visit the USGS Digital Backyard for more information.

Watershed/Stream Flow Information -

Find the Watershed for this location using the U.S. Environmental Protection Agency's site.

Climate Data Sources -

Precipitation frequency results are based on data from a variety of sources, but largely NCDC. The following links provide general information
about observing sites in the area, regardless of if their data was used in this study. For detailed information about the stations used in this study,
please refer to our documentation.

Using the **National Climatic Data Center's (NCDC)** station search engine, locate other climate stations within:

+/-30 minutes ...OR... +/-1 degree of this location (40.84902/-112.75142). Digital ASCII data can be obtained directly from NCDC.

Find Natural Resources Conservation Service (NRCS) SNOTEL (SNOwpack TELemetry) stations by visiting the **Western Regional Climate Center's state-specific SNOTEL station maps**.

Hydrometeorological Design Studies Center
DOC/NOAA/National Weather Service
1325 East-West Highway
Silver Spring, MD 20910

(301) 713-1669
Questions?: HDSC.Questions@noaa.gov

Disclaimer

SALTAIR SALT PLANT, UTAH (427578)

'riod of Record Monthly Climate Summary

Period of Record : 5/ 7/1956 to 8/31/1991

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	33.9	40.7	49.0	58.3	68.8	80.3	89.6	87.2	76.4	62.3	48.8	37.2	61.0
Average Min. Temperature (F)	17.8	23.3	31.1	38.8	47.1	56.1	63.9	61.6	51.1	39.8	30.1	21.6	40.2
Average Total Precipitation (in.)	0.71	0.75	1.31	1.73	1.70	1.02	0.68	0.78	1.21	1.32	1.11	0.82	13.15
Average Total SnowFall (in.)	5.6	3.7	3.4	1.4	0.1	0.0	0.0	0.0	0.0	0.9	2.2	6.2	23.6
Average Snow Depth (in.)	2	1	0	0	0	0	0	0	0	0	0	1	0

Percent of possible observations for period of record.
Max. Temp.: 87.2% Min. Temp.: 87.9% Precipitation: 99.7% Snowfall: 96.8% Snow Depth: 94.8%
Check Station Metadata or Metadata graphics for more detail about data completeness.

Western Regional Climate Center, wrcc@dri.edu

DUGWAY, UTAH (422257)

‑riod of Record Monthly Climate Summary

Period of Record : 9/21/1950 to 3/31/2004

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	38.2	45.0	54.1	63.1	73.8	85.1	94.7	92.1	81.2	67.2	50.6	39.5	65.4
Average Min. Temperature (F)	16.0	22.5	28.5	35.4	44.1	53.1	61.2	59.4	48.1	35.8	25.7	17.7	37.3
Average Total Precipitation (in.)	0.55	0.62	0.75	0.81	0.98	0.55	0.50	0.59	0.59	0.70	0.56	0.56	7.74
Average Total SnowFall (in.)	3.8	3.0	2.3	0.8	0.2	0.0	0.0	0.0	0.0	0.1	1.7	3.5	15.5
Average Snow Depth (in.)	1	0	0	0	0	0	0	0	0	0	0	0	0

Percent of possible observations for period of record.
Max. Temp.: 97.8% Min. Temp.: 97.8% Precipitation: 97.6% Snowfall: 96.8% Snow Depth: 89.4%
Check Station Metadata or Metadata graphics for more detail about data completeness.

Western Regional Climate Center, wrcc@dri.edu

LEACHATE COLLECTION S

1. Determine the required geonet transmissivity to provide sufficient capacity to conduct the leachate to the leachate collection pipes.

a. Bearing pressure over the geonet.

The Normal Bearing Pressure (P'):

240' Waste at 80 pcf	= 19,200 psf
2 + 2' Soil Protective Cover at 120 pcf	= 490 psf
	= 19,690 psf (use 19,700 psf)
N TOTAL	= 136.8 psi

b. Required Geonet Capacity

the geonet will be required to conduct the greatest amount of water at the low side of the planar slopes just prior to discharging leachate into the leachate collection pipes. The boundary conditions for the geonet (from top to bottom) are:

Closure and Waste Loading
2' protective soil cover comprised of fine sands and silts
8 oz. Non-woven geotextile filter fabric
Geonet
60-mil HDPE geomembrane liner

The longest one-foot wide flow path within the geonet is approximately 140 feet along the resultant slope of the wider planar surfaces. The leachate rate from this flow path length is present below.

The peak daily leachate rate to the geonet drainage layer is 0.242 inches/day based on the HELP model output. The peak daily flow from the longest flow path is calculated below.

$$q_{leachate} = (140\text{ ft})(0.242\text{ inches/day})(1\text{ foot}/\ 12\text{ inches})$$
$$q_{leachate} = 2.82\text{ ft}^3/\text{ft-day}$$

The minimum slope for the planar surfaces for the geonet after applying the projected differential settlement is 2.0%. A steeper slope will provide a more conservative design.

The required transmissivity for the geonet is given by $q_{req'd}$ and is related to the leachate rate $q_{leachate}$ by applying necessary safety factors. The combination of all the necessary safety factors is a resulting safety factor. Therefore,

$$q_{req'd} = q_{leachate} \times SF_{RES}$$

"Designing with Geosynthetics" by Robert Koerner provides recommended safety factors in the design of geonets as follows:

SF_{IN} = Safety factor for intrusion of adjacent geosynthetic materials into the geonet (1.5)
SF_{CR} = Safety factor for creep deformation of the geonet (1.5)
SF_{BCC} = Safety factor for biological and chemical clogging (2.0)

In addition to the safety factors presented above, Koerner recommends a safety factor for the design-by-function concept (SF_{DBF} = 1.5) which is a ratio of the allowable test value for the geonet to the required design value.

Combining all of the safety factors presented yields a resulting safety factor of:

$SF_{RES} = 1.5 \text{ X } 1.5 \times 2.0 \times 1.5 = 6.75$

Using the information presented above, the required geonet transmissivity is:

$(2.82)(6.75) = (\Theta \text{ m}^2/\text{sec})(10.7639 \text{ ft}^2/\text{m}^2)(86400 \text{ sec/day})(0.02)$

Where Θ is the hydraulic transmissivity of the drainage net in m^2/sec

Therefore, $\Theta = 1.023 \times 10^{-3}$ m^2/sec

Therefore the drainage net should have be tested to provide the required hydraulic transmissivity at the loading and boundary conditions provided.

c. Results of Help Model

Results of the HELP Model

Scenario	Peak Daily Leachate Drainage Geonet (in.)	Average Annual Leachate Drainage Geonet (in.)
No Waste	0.13877	1.61251
10' Waste	0.21503	2.70216
50' Waste	0.20878	2.70228
100' Waste	0.24152	2.70227
200' Waste	0.22244	2.70228

2. Determine the required capacity and diameter for the drainage pipe extending up the valleys in the floor formed by the planar floor surfaces.

a. The widest drainage area contributing leachate to the leachate collection pipes is 280 feet along the center pipe extending west from the center of the sumps. Determine the maximum length of various pipe diameters that can be placed along the 280 feet wide section of the floor with adequately capacity to convey the peak day leachate volume of 0.242 inch per day.

Area = 280 ft^2/ft of pipe length
Q = (280 ft^2)(0.242 in/day)(1 ft/12in)
Q = 5.65 ft^3/day/ft = 0.0039 ft^3/min/ft = 0.000065 ft^3/sec/ft
Q = 0.029 gpm/ft

b. Max pipe capacity: Assume 4-inch, 6-inch, and 8-inch diameter corrugated polyethylene pipe on a 1% slope after projected potential differential settlement.

Manning's n = 0.016 ("ADS Specifier Manual - Civil Engineer", Advanced Drainage Systems, Inc.)

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

Pipe Diameter (In)	Pipe Area (ft^2)	Hydraulic Radius (ft)	Flow Capacity (cfs)	Flow Capacity (gpm)	Pipe Length (ft)
3	0.20	0.79	1.25	9	321
4	0.35	1.05	2.68	20	692
6	0.79	1.57	7.91	59	2,039
8	1.40	2.09	17.03	127	4,392
10	2.18	2.62	30.87	231	7,963

6-inch diameter pipe may be used for the upper 2,000 feet of each phase area and 8-inch diameter pipe for the rest of the system to the sumps. Since the cost difference is low, use 8-inch diameter pipe for the entire length of the leachate conveyance piping.

GEOTEXTILE FILTER FA

I. Geotextile filter fabric is to be placed on top of the drainage net to serve as a filter for the overlying materials. Check design criteria of Table 3-3 p3-30 "Geotextile Engineering Manual" by U.S. Department of Transportation" to determine the soil retention and permeability criteria that must be met.

A. Native Soil Properties will be used to design the filter fabric. Other materials may be used a cover soil, however due to the high fines content of the native materials they will lead to a more conservative design. Permeability is the exception in that a higher permeability of the cover soil is more conservative. Therefore the conductivity will based on the highest cover soil conductivity that might be encountered.

B. Soil Retention

A sieve analysis of the native soil was performed by Kleinfelder[1] on the native soil. The results of this analysis are presented below in Table 1 and Figure 1. From Figure 1 the following soil parameters were estimated.

$D_{10} = 0.01$
$D_{60} = 0.12$

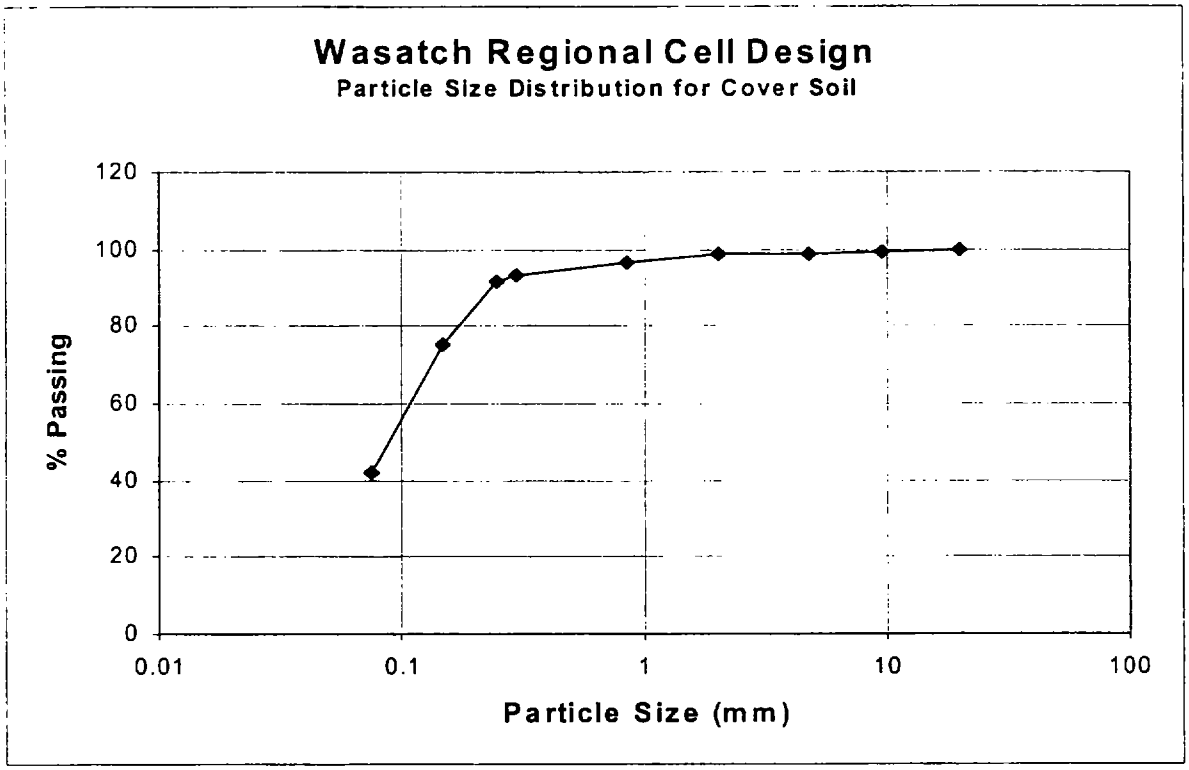
$C_u = D_{60} / D_{10}$
$C_u = 12$

$D_{85} = 0.2$ mm

Table 1

Sieve #	Size (mm)	% Finer
3/4"	20	100
3/8"	9.525	99.5
4	4.75	99
10	2	98.5
20	0.85	96.5
40	0.3	93.5
60	0.25	91.5
100	0.15	75.5
200	0.075	42

[1]Kleinfelder Lab results



Criteria from Table 3-3 of design manual for:

$\leq$ 50% passing the #200 sieve.

AOS (O_{95}) = EOS $\leq B^*D_{85\,(soil)}$

where: $B = 1$ for $C_u \leq 2$ or $C_u \geq 8$

$B = 0.5C_u$ for $2 \leq C_u \leq 4$

$B = 8/C_u$ for $4 < C_u < 8$

and:

$$C_u = D_{60\,(soil)}/D_{10\,(soil)}$$

Since C_u is greater than 8 for the native soil.

$B = 1$

EOS $\leq D_{85}$

EOS $\leq$ 0.2 mm (approx. sieve #80)

C. Permeability Criteria

$$k_{v\,(fabric)} \geq 10^{*}k_{v\,(soil)}$$
$$k_{v\,(fabric)} \geq 10^{*}\,(10^{-3}\text{cm/sec})$$
$$k_{v\,(fabric)} \geq 10^{-2}\text{ cm/sec}$$

III. Check the strength of the Filter Fabric against Burst Resistance. Since the geotextile fabric is being placed on the geonet, the fabric must have sufficient strength to bridge the ridges of the geonet without failure. According to Robert M. Koerner (1990) in "Designing with Geosynthetics" (published by Prentice-Hall, Inc.) the required fabric burst strength to bridge the gap is:

$$T_{req'd} = p'd_v$$

where

$T_{req'd}$	=	the required fabric strength
p'	=	the stress at the fabric's surface, which in the worst case would equal the overburden stress at closure
d_v	=	the maximum void diameter, or in this case the gap distance between ridges of the geonet = 0.4 inches

The Normal Bearing Pressure (P'):

250' Waste at 80 pcf	= 20,000 psf
2 + 2' Soil Protective Cover at 120 pcf	= 480 psf
	20,480 psf
N TOTAL	= 142.22 psi

Thus, $T_{req'd} = (142.22)(0.4) = 56.9$ psi

The geotextile will be designed using the design-by-function concept recommended by EPA for the design of hazardous waste facilities. According to EPA seminar publication Requirements for Hazardous Waste Landfill Design, Construction, and Closure (1989, pg. 56), "whatever parameter of a specific material one is evaluating, a required value for the material must be found using a design model and an allowable value for the material must be determined by a test method. The allowable value divided by the required value yields the design ratio, or the resulting factor of safety." Thus in evaluating the tensile strength requirement for the filter fabric, an allowable tensile strength is divided by the required tensile strength to determine the factor of safety for the design, or:

Factor of Safety (FS) = $T_{allow}/T_{req'd}$

where

T_{allow} = the allowable tensile strength as obtained from laboratory testing, and

$T_{req'd}$ = the required tensile strength as obtained from design of the actual system

Koerner (1990) in "Designing with Geosynthetics" suggests that additional factors of safety be applies to the tensile strength value found by test method to account for installation damage, creep and for biological and chemical degradation. In accordance with the procedures recommended by Koerner (1190), an additional factor of safety of 1.2 will be applied to the tensile strength found by test method for installation damage, an additional factor of safety of 1.2 will be applied to the tensile strength value for creep, and an additional factor of safety of 1.5 will be applied to test tensile strength for potential biological and chemical degradation. This value becomes the allowable value to be used in the equation above. This is in addition to the factor of safety to be used in the design-by-function concept discussed above. Thus,

$$T_{allow} = \frac{T_{given}}{(1.2x1.2x1.5)} = \frac{t_{given}}{2.16}\frac{lbs}{ft^2}$$

Assuming a design-by-function FS of 2 then

$2 = T_{allow}/T_{req'd}$
$T_{given}/2.16 = 2 * T_{req'd}$
$T_{given} = 2 * 2.16 * T_{req'd}$
$T_{given} = 2 * 2.16 * 56.9$ psi
$T_{given} = 245.8$ psi

This T_{given} was determined based on the full 250 feet of waste. Since that will not be the case over the entire landfill, the following T_{given} of 200 psi will result in a waste height of:

200 psi $= T_{given}$
$T_{req'd} = T_{given}/ (2 * 2.16)$
$T_{req'd} = 200 / (2 * 2.16)$
$T_{req'd} = 46.29$ psi

And since $T_{req'd} = p'd_v$ where $d_v = 0.4$ inches

$p' = T_{req'd}/d_v$
$p' = 46.29/0.4$
$p' = 115.7$ psi $= 16{,}666.7$ psf

Subtracting out the Soil Protective Cover

Waste Bearing Pressure = 16,666.7 -480 = 16,186.7 psf

The waste height, assuming 80 psf for the waste is

Waste Height = 16,186.7/80 = 202.3 ft

Therefore, where the waste height does not exceed 200 feet, a geosynthetic meeting 200 psi for T_{given} may be used.

IV. Koerner (1990) also defines another process acting on the fabric at the same time as the tendency to burst. This is one of tensile stress being mobilized by in-place deformation. This would occur when the geotextile fabric is locked into position by the soil above it and the ridges of the geonet below it. A lateral or in-place stress could be mobilized if two ridges of the geonet were to give or spread outward from the load of the soil placed on top. The maximum strain would occur if the ridges folded over completely, thus stressing the filter fabric. This maximum strain would be equal to the height of the ridges divided by the original gap separation. The height of each ridge is approximately 0.3 inches. The gap separation between the ridges in 0.4 inches. Thus, the maximum strain would be 0.3/0.4 = 0.75 or 75%. Koerner defines the tensile force being mobilized as being related to the pressure exerted on the fabric as follows:

$$T_{req'd} = p'(e)^2$$

$T_{req'd}$	=	the mobilized tensile force
p'	=	the applied pressure which would equal the overburden stress at closure = 142.2 psi.
e	=	the strain of the geotextile between contact points,
	=	0.75

Thus, $T_{req'd} = 142.2(0.75)^2 = 80.0$ psf for the 250 ft waste
and $T_{req'd} = 115.7(0.75)^2 = 65.1$ psf

To determine the factor of safety (FS), $T_{req'd}$ is compared with an allowable T which is the grab strength divided by the additional factors of safety referred to above.

$$T_{allow} = \frac{T_{given}}{(1.2x1.2x1.5)} = \frac{T_{given}}{2.16}\frac{lbs}{ft^2}$$

Assuming a FS of 2, then:

For the 250 ft requirement:

$2 = T_{allow}/T_{req'd}$
$T_{given}/2.16 = 2 * T_{req'd}$
$T_{given} = 2 * 2.16 * T_{req'd}$
$T_{given} = 2 * 2.16 * 80.0$ psf
$T_{given} = 345.6$ psf

For the 200 ft requirement:

$2 = T_{allow}/T_{req'd}$
$T_{given}/2.16 = 2 * T_{req'd}$
$T_{given} = 2 * 2.16 * T_{req'd}$
$T_{given} = 2 * 2.16 * 65.1$ psf
$T_{given} = 281.2$ psf

SUMP CAPACITY

HANSEN ALLEN & LUCE inc ENGINEERS	CLIENT:	Wasatch Regional	SHEET 1	OF 1
	PROJECT:	Landfill Permit	COMPUTED:	GLJ
	FEATURE:	Sump Capacity Calculation	CHECKED:	KCS
	PROJECT NO.:	113.30.100	DATE: September 2004	

I. Determine the sump capacity.

Surface Area$_{top}$ = 3,200 ft^2

Surface Area$_{bottom}$ = 2,756 ft^2

Surface Area$_{average}$ = (3200 + 2756) /2 = 2,978.2 ft^2

Average Depth = (2.5 + 0.6) /2 = 1.6 ft

Total Volume = 2978.2 * 1.6 = 4,765.1 ft^3

Total 8" pipe length = 105.4 ft
Total 24" pipe length = 7.8 ft
8" Pipe Cross Sectional Area = pi*(4/12)^2 = 0.349 ft^2
24" Pipe Cross Sectional Area = pi*(12/12)^2= 3.14 ft^2

Total Pipe Volume = 105.4*0.349 + 7.8*3.14= 61.3 ft^3

The rock porosity will be assumed to be 0.32

Rock Volume = 4765.1 - 61.3 = 4,703.8 ft^3

Net Volume = 4,703.8*(0.32) + 61.3 = 1,566.5 ft^3

GCL HYDRAULIC COMPA

I. Determine GCL Compatibility with by looking at both hydraulic issues and the HELP model.

A. **Hydraulic Issues**

One of the critical issues associated with use of a GCL is its ability to minimize the potential of contamination to ground water from migration of leachate water through the lining system as compared to a compacted clay liner. According to a technical paper titled *Technical Equivalency Assessment of GCL's To CCL's* prepared by R.M. Koerner from the Geosynthetic Research Institute, Drexel University and D.E. Daniel from University of Texas at Austin, a hydraulic comparison can best be demonstrated by an application of Darcy's law.

$V = k((H+T)/T)$ where: $k =$ hydraulic conductivity
$H =$ depth of liquid ponded on the liner
$T =$ thickness of the liner

In order to establish equivalency between the GCL and a CCL:

$V_{GCL} = V_{CCL}$ or

$k_{GCL}((H+T_{GCL})/T_{GCL}) = k_{CCL}((H+T_{CCL})/T_{CCL})$

Substituting in the values of T for the GCL and the values of k and T for the CCL (H is assumed constant), the equation can be solved for and equivalent k required for the GCL. Assuming $k_{CCL} = 1 \times 10\text{-}7$ cm/sec, $T_{CCL} = 2$ feet or about 600 mm and $T_{GCL} = 7$ mm after hydration, $k_{GCL} = 3.4 \times 10\text{-}9$ cm/sec. This is consistent with the hydraulic conductivity of the GCL materials.

$$(3.4E-9cm/\sec)\cdot(\frac{H+0.7cm}{0.7cm}) = (1E-7cm/\sec)\cdot(\frac{H+60cm}{60cm})$$

$$H = 30.3cm = 1ft$$

As can be seen from the comparative analysis presented above, a single GCL is hydraulically equivalent under steady state flow conditions to the two feet of compacted clay liner when the ponding depth is around 1 ft. Completely replacing two feet of compacted clay with a GCL will provide hydraulic equivalence in providing for ground water contamination protection.

B. **HELP Model**

EPA's Hydrologic Evaluation of Landfill Performance (HELP) model was used previously to model percolation of precipitation water through the lining systems of the current design concept in the floor area. Additional modeling was performed to model percolation of precipitation water through the proposed design concept in the floor area. The results of the

HELP models were compared to provide justification for the proposed lining system. The proposed system should provide an equivalent or better lining system for protection of ground water.

Precipitation, daily temperature, evapotranspiration, and solar radiation data used for modeling of the current system were used for the proposed lining system. The only change to the model was to the bottom layer. The GCL in the current design was changed to a two foot thick CCL with a saturated hydraulic conductivity of 1.0E-7 cm/sec.

Results from the model estimate an average annual leakage rate through the bottom lining system of about 0.169 cubic foot per year for the current design using a GCL and 0.375 cubic foot per year under the design using a CCL. Based on the results from the HELP model, the modified concept provides a reduced estimate of leakage through the bottom lining system.

Client: **Allied Waste**
Project: **Wasatch Regional**
Feature: **GCL Equivalency**
Project No.: **113.30.100**

Determine: The hydraulic equivalency of Geosynthetic Clay Liners (GCL) to Compacted Clay Liners (CCL)

Darcy's Law provides: V = k((H+T)/T)

where: V =
k = hydraulic conductivity of liner material
H = depth of liquid ponded on liner material
T = thickness of liner material

Determine V_{CCL}

H_{CCL} = 1.0 ft = 30.48 cm, maximum allowable hydraulic head on the liner outside the sump area
k_{CCL} = 1.0E-07 cm/sec
T_{CCL} = 2.0 ft = 60.96 cm, minimum required thickness at a permeability of 1×10^{-7} cm/sec

Therefore, V_{CCL} = 1.5E-07 cm/sec

Determine V_{GCL}

Tabulate a relationship between k_{GCL} and T_{GCL} as variables to provide equivalency between V_{CCL} and V_{GCL}. T_{GCL} is a hydrated thickness for the GCL material.

H_{GCL} = 1 ft = 30.48 cm, maximum allowable hydraulic head on the liner outside the sump area

k_{GCL}	T_{GCL}		
(cm/sec)	(mm)	(cm)	(in)
1.9E-09	4.0	0.40	0.157
2.4E-09	5.0	0.50	0.197
2.9E-09	6.0	0.60	0.236
3.4E-09	7.0	0.70	0.276
3.8E-09	8.0	0.80	0.315
4.3E-09	9.0	0.90	0.354
4.8E-09	10.0	1.00	0.394
5.2E-09	11.0	1.10	0.433
5.7E-09	12.0	1.20	0.472
6.1E-09	13.0	1.30	0.512
6.6E-09	14.0	1.40	0.551

INDEX FLUX AND PERMEABILITY OF GCL's
TEST RESULTS
ASTM D-5887 / D-5084 / EPA 9100

JLT

Client	:	CETCO	Date	:	03-13-04
Project Location	:	Toole Landfill, Utah	Job No.	:	04LG352.01
Sample Number	:	Roll : 516 Lot 200405FA	Tested By	:	HT
Description	:	Bentomat SDN	Checked By	:	JB

Permeant Fluid : Leachate Provided by Client

Physical Property Data

		Initial				Final
Initial Clay Height (in)	:	0.20		Final Height of Clay (in)	:	0.25
Initial Diameter (in)	:	4.00		Final Diameter of Clay (in)	:	4.00
Initial Wet Weight (g)	:	47.20		Final Wet Weight(Clay) (g)	:	69.20
Wet Density (pcf)	:	71.48		Wet Density (pcf)	:	83.84
Moisture Content %	:	22.00	Est	Moisture Content %	:	112.90
Dry Density (pcf)	:	58.59		Dry Density (pcf)	:	39.38

Test Parameters

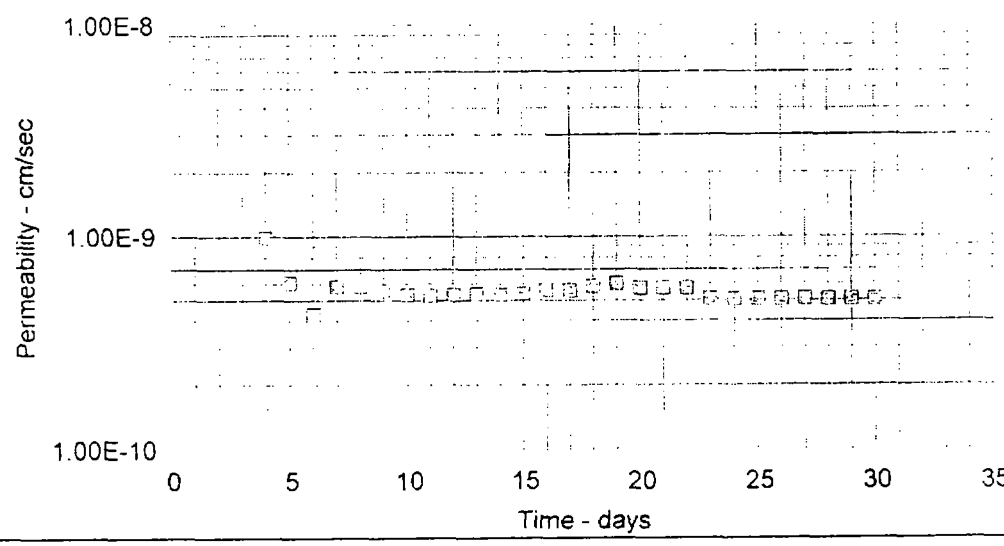
Fluid	:	Site Leachate	**Effective**		
Cell Pressure psi)	:	80.00	**Confining Pressure (psi)**	:	**4**
Head Water (psi)	:	77.00	Gradient	:	220.80
Tail Water (psi)	:	75.00	Head Differential (psi)	:	2

Flux and Permeability Input Data

Minimum Saturation Time is 48 hours

Area, A = 0.00811 m^2
Thickness, t = 0.25 in

Total Inflow to date : 16 9 cc



JLT Laboratories, Inc. 938 S Central Ave. Canonsburg, Pa 15317 Tel 724-746-4441 . Fax 724-745-4261

Daily Readings and Computations

JLT

Client : CETCO
Project Location : Toole Landfill, Utah
Sample Number : Roll : 516 Lot 200405FA
Description : Bentomat SDN

Date : 03-13-04
Job No. : 04LG352.01
Tested By : HT
Checked By : JB

Days	Date	Flow	Time	Elapsed	Flux	k	Cum Inflow
		cc	min	Time (sec)	(m^3/m^2)/sec	cm/sec	cc
1	02/13/2004	48 hours of hydration per ASTM					
2	02/14/2004						
3	02/15/2004	0.00	0	0			0.0
4	02/16/2004	3.90	1442	86520	5.56E-009	9.89E-010	3.9
5	02/17/2004	2.40	1441	86460	3.42E-009	6.09E-010	6.3
6	02/18/2004	1.70	1445	86700	2.42E-009	4.30E-010	8.0
7	02/19/2004	2.30	1444	86640	3.27E-009	5.82E-010	10.3
8	02/20/2004	2.30	1442	86520	3.28E-009	5.83E-010	12.6
9	02/21/2004	2.20	1443	86580	3.13E-009	5.57E-010	14.8
10	02/22/2004	2.10	1440	86400	3.00E-009	5.33E-010	16.9
11	02/23/2004	2.00	1388	83280	2.96E-009	5.27E-010	18.9
12	02/24/2004	1.90	1310	78600	2.98E-009	5.30E-010	20.8
13	02/25/2004	2.10	1439	86340	3.00E-009	5.33E-010	22.9
14	02/26/2004	2.10	1445	86700	2.99E-009	5.31E-010	25.0
15	02/27/2004	2.20	1501	90060	3.01E-009	5.36E-010	27.2
16	02/28/2004	2.20	1442	86520	3.14E-009	5.58E-010	29.4
17	02/29/2004	2.20	1445	86700	3.13E-009	5.56E-010	31.6
18	03/01/2004	2.30	1442	86520	3.28E-009	5.83E-010	33.9
19	03/02/2004	2.25	1368	82080	3.38E-009	6.01E-010	36.2
20	03/03/2004	2.25	1441	86460	3.21E-009	5.71E-010	38.4
21	03/04/2004	2.30	1475	88500	3.21E-009	5.70E-010	40.7
22	03/05/2004	2.25	1442	86520	3.21E-009	5.70E-010	43.0
23	03/06/2004	2.00	1440	86400	2.86E-009	5.08E-010	45.0
24	03/07/2004	2.00	1441	86460	2.85E-009	5.07E-010	47.0
25	03/08/2004	2.00	1439	86340	2.86E-009	5.08E-010	49.0
26	03/09/2004	2.00	1443	86580	2.85E-009	5.07E-010	51.0
27	03/10/2004	2.00	1437	86220	2.86E-009	5.09E-010	53.0
28	03/11/2004	2.00	1444	86640	2.85E-009	5.06E-010	55.0
29	03/12/2004	2.00	1442	86520	2.85E-009	5.07E-010	57.0
30	03/13/2004	2.00	1447	86820	2.84E-009	5.05E-010	59.0

WASTE RUNOFF CONTAI

Purpose: To determine the capacity requirements for runoff containment within the active landfill.

Method: The SCS curve number method as described in Technical Release No. 55.

Required: In order to calculate the runoff volume, the following steps and information are required:

- Tributary area contributing to runoff.
- A Representative Soil Conservation Service(SCS) curve number (CN).
- 25-year 24-hour precipitation depth as required by regulation.

Delineation: Runoff will be determined based on the volume generated per acre of open and active cell area.

Curve Numbers: The curve number was determined based assumptions made for the daily cover to be used during landfill operation. The soil used for daily cover will consist of on-site soils and are of the type B hydrologic soil group based on information and soils defined in the NRCS study "Soil Survey of Tooele Area, Utah." Table 2-2d of Technical Release 55 provides a curve number of 82 for dirt road type conditions (including the right-of-way) with type B soils. Daily cover soils are placed and compacted using a dozer or landfill compactor type equipment that leaves an irregular surface that will provide additional interception storage beyond that of a dirt road and probably beyond that of a dirt road plus the right-of-way because of the individual ponding areas provided by the equipment. Using a curve number of 82 should provide representative, but conservative, results for the daily cover material.

Precipitation: Design for the 25-year 24-hour precipitation event is required by regulations for MSWLF's. The rainfall amounts were taken from the "Point Precipitation Frequency Estimates from NOAA Atlas 14". The precipitation depth value used is 2.06 inches.

Calculations:

Rainfall runoff depth (Q) is determined by:

$Q = ((P-0.2S)^2)/(P+0.8S)$ Where: Q = Runoff depth (inches)
P = Precipitation depth (inches)
S = Potential maximum retention after runoff begins (inches) = (Ia)/(0.2)
Where Ia = Initial abstraction (inches)

Also S is related the SCS curve number (CN) as follows:

$S = (1000/CN)-10$

Determine Runoff Depth Per Acre of Area

$S = (1000/82)-10 = 2.20$

$Q = ((2.06-0.2(2.2))^2)/(2.06+0.8(2.2)) = 0.69$ inches

Runoff quantity per acre is 0.69/12 = 0.06 acre foot per acre = 2,613 cf/acre

Conclusion:

Required runoff containment capacity is, therefore, 0.06 acre foot (2,613 cf) per acre of open cell area. Therefore, for the first phase of construction the containment capacity for approximately 20 to 22 acres is 1.2 to 1.32 acre-feet (52,272 to 57,500 cf). This containment capacity may be provided in a number of ways including:

- A waste set-back from the inside slope of the cell.
- A ponding area on the waste surface.
- Ditches between the waste and the interior slope of the cells.
- Providing separate lined runoff containment storage areas.
- A combination of the above or any other method that will provide the required containment capacity.

Runoff water may be used inside the lined cell areas for dust control and compaction.

We recommend that facility operators provide a minimum of two feet freeboard within all containment areas provided.

POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

Utah 40.85579°N 112.75219°W 4435 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 3
G.M. Bonnin, D. Todd, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2003
Extracted: Thu Nov 18 2004

Precipitation Frequency Estimates (inches)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	0.14	0.22	0.27	0.37	0.45	0.56	0.63	0.80	0.99	1.27	1.45	1.64	1.85	2.06	2.58	3.05	3.67	4.30
5	0.20	0.30	0.38	0.51	0.63	0.74	0.81	0.98	1.21	1.54	1.77	2.02	2.28	2.51	3.12	3.70	4.41	5.16
10	0.25	0.38	0.47	0.64	0.79	0.90	0.96	1.14	1.38	1.76	2.04	2.33	2.62	2.87	3.54	4.21	4.97	5.81
25	0.33	0.51	0.63	0.84	1.04	1.16	1.21	1.38	1.64	2.06	2.40	2.77	3.09	3.36	4.08	4.87	5.69	6.64
50	0.40	0.62	0.76	1.03	1.27	1.40	1.43	1.57	1.84	2.29	2.69	3.12	3.45	3.73	4.47	5.37	6.21	7.23
100	0.49	0.75	0.93	1.25	1.54	1.67	1.70	1.80	2.06	2.52	2.98	3.48	3.83	4.11	4.87	5.86	6.72	7.81
200	0.59	0.90	1.11	1.50	1.86	1.99	2.02	2.09	2.31	2.75	3.29	3.86	4.21	4.49	5.24	6.34	7.19	8.34
500	0.75	1.14	1.41	1.90	2.35	2.50	2.52	2.59	2.75	3.08	3.70	4.38	4.73	4.98	5.72	6.96	7.79	9.01
1000	0.89	1.35	1.68	2.26	2.80	2.95	2.97	3.03	3.13	3.36	4.01	4.79	5.13	5.36	6.06	7.41	8.21	9.47

Text version of table

* These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval. Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.

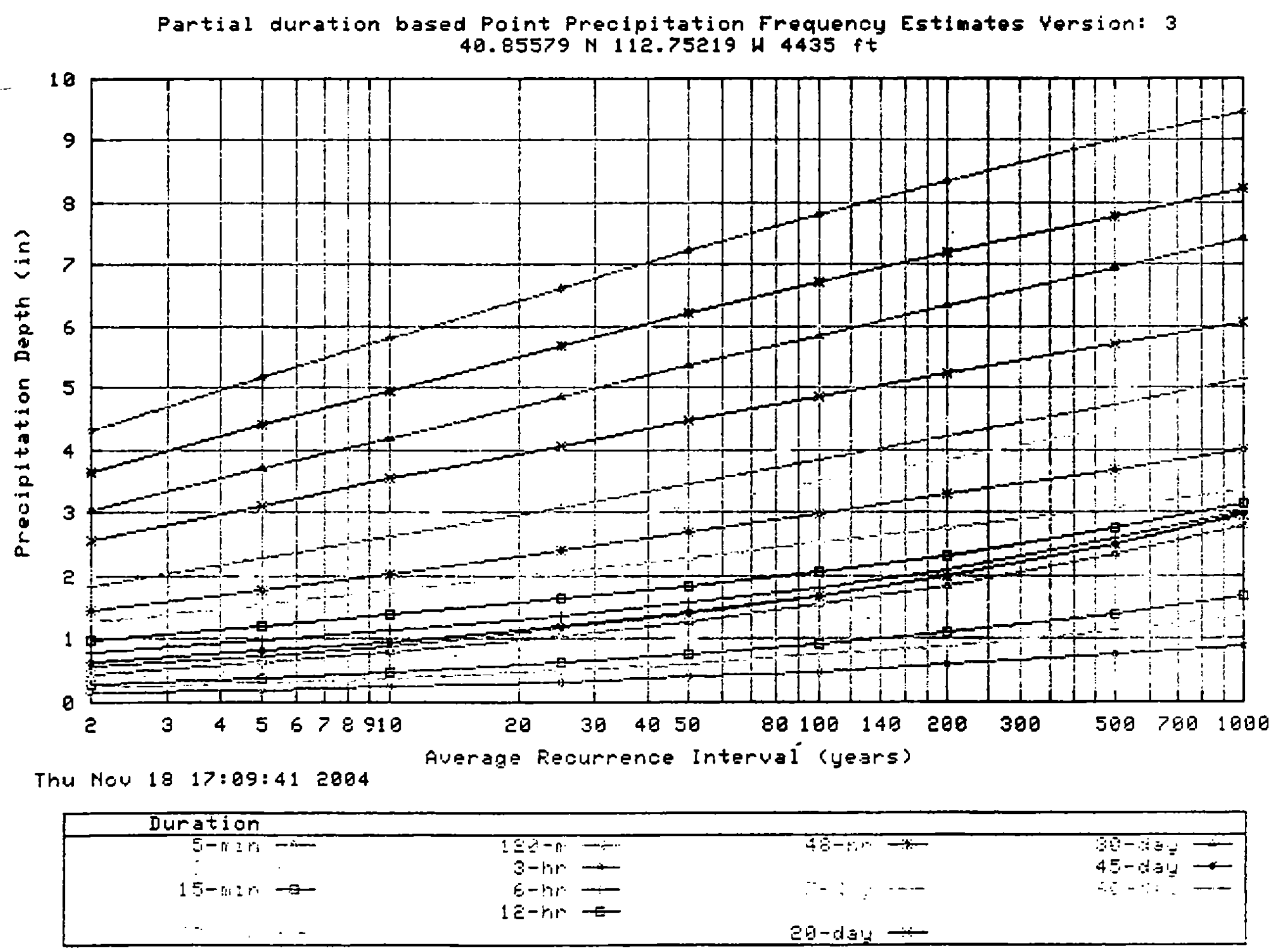


Table 2-2a Runoff curve numbers for urban areas [1]

Cover description: Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.

2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

APPENDIX E

STORM WATER MANAGEMENT DESIGN CALCULATIONS

HYDROLOGY FOR RUN-ON STORM WATER

STORM WATER CONVEYANCE AND RIPRAP DESIGN

CLOSURE HYDROLOGY

CLOSURE HYDRAULIC DESIGN

CLOSURE EROSION PROTECTION

HYDROLOGY FOR RUN-ON ST[illegible]ER

HANSEN ALLEN & LUCE inc ENGINEERS	CLIENT: Wasatch Regional PROJECT: Landfill Permit FEATURE: Hydrology PROJECT NO.: 113.30.100	SHEET 1 OF 2 COMPUTED: GLJ CHECKED: KCS DATE: October 2004

Purpose: To determine the design flows to use for the channels around the facility.

Method: The SCS curve number method was used with the HEC-1 hydrology model. The HEC-1 model was set up using the HAL Water Suite.

Required: In order to calculate the runoff the following steps and information are required:

- A delineation of the tributary area.
- A weighted or representative Soil Conservation Service(SCS) curve number (CN) for the tributary area.
- Lag time.
- Storm Distribution.
- 100 year-24 hour precipitation.
- Areal reduction factor.

Delineation: The delineation of the subbasins, shown in Figure 1, was based on the contours provided on the USGS quad maps. There will be two channels designed to divert runoff around the facility, one that will direct flow to the north and the other to the south. Subbasins were divided along the channel routes in order to allow for a progressive design instead of designing the entire channel for the final maximum flow.

Curve Numbers: The curve numbers were determined based on the hydrologic soil type and soil cover as shown in Figure 2. The soil vegetation cover and conditions were assumed based on information given in the NRCS study "Soil Survey of Tooele Area, Utah" and verified by a field visit on October 26, 2004. The cover conditions were combined with the hydrologic soil type to produce a curve number based on Table 2-2d of Technical Release 55. Because each subbasin contained several different soil types and covers, a weighted curve number was applied to each subbasin based on area. The calculations of the weighted curve numbers are entitled "Weighted Hydrologic Curve Numbers."

Precipitation: A 100 year - 24 hour event was used for the design. The rainfall amount was taken from the "Point Precipitation Frequency Estimates from NOAA Atlas 14". One precipitation value was used for all of the subbasins.

Storm Distribution: The distribution used for the 24-hour event was the SCS Type II.

Lag Time: The lag times were calculated by using the Time of Concentration and the equation $T_L = 0.6Tc$. Tc was calculated using Worksheet 3 in TR-55. A calculation sheet for each subbasin is provided and are labeled with their subbasin name.

Areal Reduction: The magnitude of the area tributary to the landfill site is large enough to warrant the use of a reduction of the precipitation values because the likelihood of the full amount hitting the whole region decreases with an increase of tributary area. The factor was based on the Salt Lake City Hydrology Manual. According to the manual, a 24-hour event has an Areal Reduction Factor of:

ARF = .01*(100-2*Area ^ .46) where the Area = 3.68 mi^2

ARF = 0.96

Results: The results of the HEC-1 model run are summarized in Figure 3 and can be found on page 25 of the HEC-1 output. The southern flow should be designed to carry 551 cfs while the northern flow should be designed for 86 cfs.

HANSEN ALLEN & LUCE inc ENGINEERS

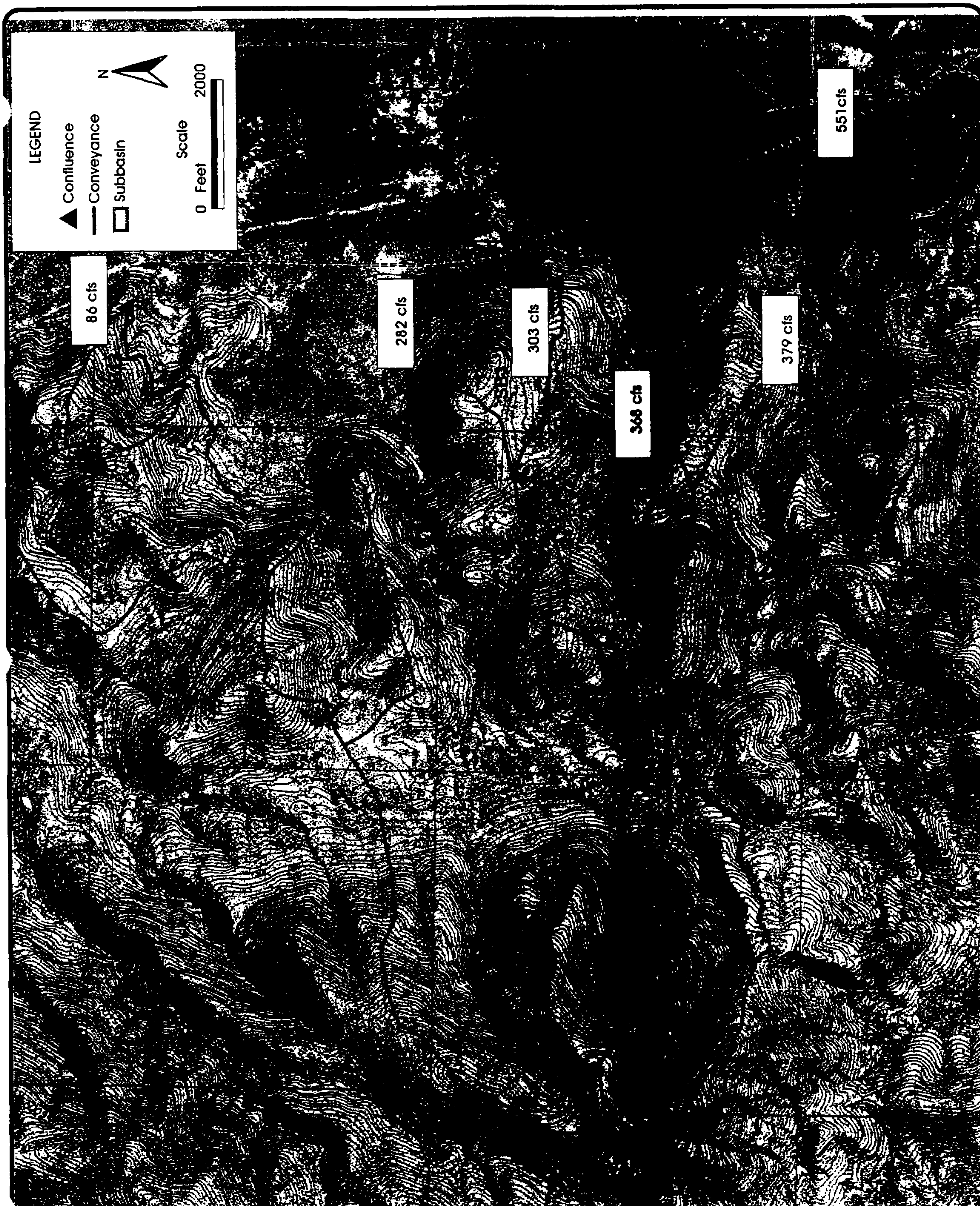
OFF-SITE HYDROLOGY MODEL

FIGURE 1

HANSEN ALLEN & LUCE inc ENGINEERS

SCS SOIL TYPE

FIGURE 2



HANSEN ALLEN & LUCE INC ENGINEERS

OFF-SITE MODEL RESULTS

FIGURE 3

```
****************************************          ****************************************
*                                      *          *                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *          *  U.S. ARMY CORPS OF ENGINEERS        *
*              JUN   1998              *          *  HYDROLOGIC ENGINEERING CENTER       *
*            VERSION 4.1               *          *        609 SECOND STREET             *
*                                      *          *     DAVIS, CALIFORNIA 95616          *
* RUN DATE   16NOV04  TIME  13:36:50   *          *        (916) 756-1104                *
*                                      *          *                                      *
****************************************          ****************************************


                    X     X  XXXXXXX   XXXXX          X
                    X     X  X        X     X        XX
                    X     X  X        X               X
                    XXXXXXX  XXXX     X        XXXXX  X
                    X     X  X        X               X
                    X     X  X        X     X         X
                    X     X  XXXXXXX   XXXXX         XXX


THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM


1                                              HEC-1 INPUT
         LINE            ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

                         *DIAGRAM
 *** FREE ***
            1            ID   Hydrology C:\gisfiles\113\30.100\Hydrology\Template.cnt
            2            ID   Allied Waste - Wasatch Regional LF Hydrology
            3            ID
            4            IT      5                 288
            5            IO      3
            6            JR   PREC    0.96

            7            KK   SB11
            8            BA  0.319
            9            PB   2.61
           10            IN     30
           11            PI      0    .005    .006    .006    .006    .006    .006    .007    .007    .007
           12            PI   .008    .008    .009    .009     .01     .01     .01    .012    .015    .016
           13            PI   .018    .023    .033    .046    .038    .072    .037    .027    .023    .018
           14            PI   .015    .013    .012    .011    .011     .01    .009    .009    .008    .008
           15            PI   .008    .008    .006    .006    .006    .005    .005    .005    .005
           16            LS      0    87.5
           17            UD   0.47
           18            KO                                 22

           19            KK    CV7
           20            RD 2185.47 0.00250   0.040         TRAP    5.00    4.00
           21            KO                                 22

           22            KK   SB13
           23            BA  0.135
           24            PB   2.61
           25            IN     30
           26            PI      0    .005    .006    .006    .006    .006    .006    .007    .007    .007
           27            PI   .008    .008    .009    .009     .01     .01     .01    .012    .015    .016
           28            PI   .018    .023    .033    .046    .038    .072    .037    .027    .023    .018
           29            PI   .015    .013    .012    .011    .011     .01    .009    .009    .008    .008
           30            PI   .008    .008    .006    .006    .006    .005    .005    .005    .005
           31            LS      0   88.27
           32            UD   0.31
           33            KO                                 22

           34            KK    HC6
           35            HC      2
           36            KO                                 22

           37            KK    SB9
           38            BA  0.236
           39            PB   2.61
           40            IN     30
           41            PI      0    .005    .006    .006    .006    .006    .006    .007    .007    .007
           42            PI   .008    .008    .009    .009     .01     .01     .01    .012    .015    .016
           43            PI   .018    .023    .033    .046    .038    .072    .037    .027    .023    .018
```

```
 44         PI    .015    .013    .012    .011    .011     .01    .009    .009    .008    .008
 45         PI    .008    .008    .006    .006    .006    .005    .005    .005    .005
 46         LS       0      89
 47         UD    0.61
 48         KO                                      22
                                            HEC-1 INPUT
1
LINE        ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


 49         KK     CV8
 50         RD 4187.50 0.00250   0.040            TRAP    5.00    4.00
 51         KO                                      22

 52         KK    SB10
 53         BA   1.345
 54         PB    2.61
 55         IN      30
 56         PI       0    .005    .006    .006    .006    .006    .006    .007    .007    .007
 57         PI    .008    .008    .009    .009     .01     .01     .01    .012    .015    .016
 58         PI    .018    .023    .033    .046    .038    .072    .037    .027    .023    .018
 59         PI    .015    .013    .012    .011    .011     .01    .009    .009    .008    .008
 60         PI    .008    .008    .006    .006    .006    .005    .005    .005    .005
 61         LS       0   86.42
 62         UD    0.46
 63         KO                                      22

 64         KK     HC8
 65         HC       2
 66         KO                                      22

 67         KK     CV1
 68         RD 2705.84 0.00250   0.040            TRAP    5.00    4.00
 69         KO                                      22

 70         KK     SB3
 71         BA   0.128
 72         PB    2.61
 73         IN      30
 74         PI       0    .005    .006    .006    .006    .006    .006    .007    .007    .007
 75         PI    .008    .008    .009    .009     .01     .01     .01    .012    .015    .016
 76         PI    .018    .023    .033    .046    .038    .072    .037    .027    .023    .018
 77         PI    .015    .013    .012    .011    .011     .01    .009    .009    .008    .008
 78         PI    .008    .008    .006    .006    .006    .005    .005    .005    .005
 79         LS       0   89.00
 80         UD    0.44
 81         KO                                      22

 82         KK     HC3
 83         HC       2
 84         KO                                      22

 85         KK     CV2
 86         RD 2294.41 0.00250   0.040            TRAP    5.00    4.00
 87         KO                                      22

 88         KK     SB2
 89         BA   0.378
 90         PB    2.61
 91         IN      30
 92         PI       0    .005    .006    .006    .006    .006    .006    .007    .007    .007
 93         PI    .008    .008    .009    .009     .01     .01     .01    .012    .015    .016
 94         PI    .018    .023    .033    .046    .038    .072    .037    .027    .023    .018
 95         PI    .015    .013    .012    .011    .011     .01    .009    .009    .008    .008
                                            HEC-1 INPUT
1
LINE        ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 96         PI    .008    .008    .006    .006    .006    .005    .005    .005    .005
 97         LS       0   87.70
 98         UD    0.70
 99         KO                                      22

100         KK     HC2
101         HC       2
102         KO                                      22

103         KK     CV3
104         RD 2667.77 0.00250   0.040            TRAP    5.00    4.00
105         KO                                      22

106         KK     SB8
107         BA   0.123
108         PB    2.61
109         IN      30
110         PI       0    .005    .006    .006    .006    .006    .006    .007    .007    .007
111         PI    .008    .008    .009    .009     .01     .01     .01    .012    .015    .016
```

```
112           PI   .018   .023   .033   .046   .038   .072   .037   .027   .023   .018
113           PI   .015   .013   .012   .011   .011    .01   .009   .009   .008   .008
114           PI   .008   .008   .006   .006   .006   .005   .005   .005   .005
115           LS      0  92.79
116           UD   0.50
117           KO                               22

118           KK    HC1
119           HC      2
120           KO                               22

121           KK    CV4
122           RD 2880.40 0.00250  0.040      TRAP   5.00   4.00
123           KO                               22

124           KK   SB12
125           BA  1.194
126           PB   2.61
127           IN     30
128           PI      0   .005   .006   .006   .006   .006   .006   .007   .007   .007
129           PI   .008   .008   .009   .009    .01    .01    .01   .012   .015   .016
130           PI   .018   .023   .033   .046   .038   .072   .037   .027   .023   .018
131           PI   .015   .013   .012   .011   .011    .01   .009   .009   .008   .008
132           PI   .008   .008   .006   .006   .006   .005   .005   .005   .005
133           LS      0  88.24
134           UD   0.49
135           KO                               22

136           KK    HC7
137           HC      2
138           KO                               22
139           ZZ
```

```
1
          SCHEMATIC DIAGRAM OF STREAM NETWORK
INPUT
 LINE     (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

  NO.     (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

   7        SB11
               V
               V
  19        CV7
               .
               .
  22           .          SB13
               .             .
               .             .
  34        HC6...............
               .
               .
  37           .          SB9
               .             V
               .             V
  49           .          CV8
               .             .
               .             .
  52           .             .          SB10
               .             .             .
               .             .             .
  64           .          HC8...............
               .             V
               .             V
  67           .          CV1
               .             .
               .             .
  70           .             .          SB3
               .             .             .
               .             .             .
  82           .          HC3...............
               .             V
               .             V
  85           .          CV2
               .             .
               .             .
  88           .             .          SB2
               .             .             .
               .             .             .
 100           .          HC2...............
               .             V
               .             V
 103           .          CV3
               .             .
               .             .
 106           .             .          SB8
               .             .             .
               .             .             .
```

```
 118          .         HC1............
              .          V
              .          V
 121          .         CV4
              .          .
              .          .
 124          .          .          SB12
              .          .             .
              .          .             .
 136          .         HC7............

(***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1***************************************
*                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *
*             JUN   1998               *
*            VERSION 4.1               *
*                                      *
*  RUN DATE   16NOV04  TIME  13:36:50  *
*                                      *
****************************************

***************************************
*                                     *
*   U.S. ARMY CORPS OF ENGINEERS      *
*   HYDROLOGIC ENGINEERING CENTER     *
*        609 SECOND STREET            *
*      DAVIS, CALIFORNIA 95616        *
*          (916) 756-1104             *
*                                     *
***************************************


                    Hydrology C:\gisfiles\113\30.100\Hydrology\Template.cnt


  5 IO          OUTPUT CONTROL VARIABLES
                      IPRNT           3  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE

    IT          HYDROGRAPH TIME DATA
                       NMIN           5  MINUTES IN COMPUTATION INTERVAL
                      IDATE     1     0  STARTING DATE
                      ITIME        0000  STARTING TIME
                         NQ         288  NUMBER OF HYDROGRAPH ORDINATES
                     NDDATE     1     0  ENDING DATE
                     NDTIME        2355  ENDING TIME
                      ICENT          19  CENTURY MARK

                  COMPUTATION INTERVAL     .08 HOURS
                       TOTAL TIME BASE   23.92 HOURS

          ENGLISH UNITS
               DRAINAGE AREA          SQUARE MILES
               PRECIPITATION DEPTH    INCHES
               LENGTH, ELEVATION      FEET
               FLOW                   CUBIC FEET PER SECOND
               STORAGE VOLUME         ACRE-FEET
               SURFACE AREA           ACRES
               TEMPERATURE            DEGREES FAHRENHEIT

    JP          MULTI-PLAN OPTION
                      NPLAN           1  NUMBER OF PLANS

    JR          MULTI-RATIO OPTION
                    RATIOS OF PRECIPITATION
                       .96


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
  7 KK    *    SB11    *
          *            *
          **************

 10 IN          TIME DATA FOR INPUT TIME SERIES
                      JXMIN          30  TIME INTERVAL IN MINUTES
                     JXDATE     1     0  STARTING DATE
                     JXTIME           0  STARTING TIME

 18 KO          OUTPUT CONTROL VARIABLES
                      IPRNT           3  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                       IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
```

TIMINT .083 TIME INTERVAL IN HOURS

SUBBASIN RUNOFF DATA

8 BA SUBBASIN CHARACTERISTICS
TAREA .32 SUBBASIN AREA

PRECIPITATION DATA

9 PB STORM 2.61 BASIN TOTAL PRECIPITATION

11 PI INCREMENTAL PRECIPITATION PATTERN

.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
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.01	.01	.01	.01	.01	.01	.01	.01	.01	.01
.01	.01	.01	.01	.01	.01	.01	.01	.01	.01
.01	.01	.00	.00	.00	.00	.00	.00	.00	.00
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.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00			

16 LS SCS LOSS RATE
STRTL .29 INITIAL ABSTRACTION
CRVNBR 87.50 CURVE NUMBER
RTIMP .00 PERCENT IMPERVIOUS AREA

17 UD SCS DIMENSIONLESS UNITGRAPH
TLAG .47 LAG

***

UNIT HYDROGRAPH
30 END-OF-PERIOD ORDINATES

22.	67.	136.	224.	283.	301.	291.	258.	215.	160.
119.	91.	71.	55.	42.	32.	25.	19.	14.	11.
8.	7.	5.	4.	3.	2.	2.	1.	1.	0.

*** *** *** *** ***

HYDROGRAPH AT STATION SB11
FOR PLAN 1, RATIO = .96

TOTAL RAINFALL = 2.61, TOTAL LOSS = 1.17, TOTAL EXCESS = 1.44

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	23.92-HR
+ (CFS)	(HR)					
		(CFS)				
+ 67.	13.25		33.	12.	12.	12.
		(INCHES)	.951	1.418	1.418	1.418
		(AC-FT)	16.	24.	24.	24.

CUMULATIVE AREA = .32 SQ MI

*** *** *** *** ***

HYDROGRAPH AT STATION SB11
FOR PLAN 1, RATIO = .96

TOTAL RAINFALL = 2.51, TOTAL LOSS = 1.15, TOTAL EXCESS = 1.35

PEAK FLOW	TIME	MAXIMUM AVERAGE FLOW			
		6-HR	24-HR	72-HR	23.92-HR
+ (CFS)	(HR)				

```
                              (CFS)
      63.       13.25                  31.          11.          11.          11.
                            (INCHES)   .894        1.330        1.330        1.330
                             (AC-FT)    15.          23.          23.          23.

                            CUMULATIVE AREA =      .32 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

              **************
              *            *
  19 KK       *     CV7    *
              *            *
              **************

  21 KO            OUTPUT CONTROL VARIABLES
                         IPRNT          3  PRINT CONTROL
                         IPLOT          0  PLOT CONTROL
                         QSCAL         0.  HYDROGRAPH PLOT SCALE
                         IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                          IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                         ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                         ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                        TIMINT       .083  TIME INTERVAL IN HOURS

                 HYDROGRAPH ROUTING DATA

  20 RD            MUSKINGUM-CUNGE CHANNEL ROUTING
                             L      2185.  CHANNEL LENGTH
                             S      .0025  SLOPE
                             N       .040  CHANNEL ROUGHNESS COEFFICIENT
                            CA        .00  CONTRIBUTING AREA
                         SHAPE       TRAP  CHANNEL SHAPE
                            WD       5.00  BOTTOM WIDTH OR DIAMETER
                             Z       4.00  SIDE SLOPE

                                                          ***
                                    COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                         COMPUTATION TIME STEP
                  ELEMENT    ALPHA      M        DT       DX       PEAK    TIME TO    VOLUME    MAXIMUM
                                                                            PEAK                CELERITY
                                               (MIN)     (FT)     (CFS)    (MIN)      (IN)      (FPS)

                   MAIN       .62      1.36     5.00    546.37    62.09    810.00     1.32      2.42

                                         INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                   MAIN       .62      1.36     5.00              62.09    810.00     1.32

CONTINUITY SUMMARY (AC-FT) - INFLOW= .2266E+02 EXCESS= .0000E+00 OUTFLOW= .2242E+02 BASIN STORAGE= .2885E+00 PERCENT ERROR=   -.2


       ***           ***           ***           ***           ***

                           HYDROGRAPH AT STATION       CV7
                             FOR PLAN 1, RATIO =   .96

  PEAK FLOW      TIME                           MAXIMUM AVERAGE FLOW
                                       6-HR        24-HR        72-HR      23.92-HR
+   (CFS)        (HR)
                              (CFS)
+       62.      13.50                  31.          11.          11.          11.
                            (INCHES)   .893        1.316        1.316        1.316
                             (AC-FT)    15.          22.          22.          22.

                            CUMULATIVE AREA =      .32 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

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              *            *
  22 KK       *    SB13    *
              *            *
              **************
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 25 IN          TIME DATA FOR INPUT TIME SERIES
                     JXMIN         30  TIME INTERVAL IN MINUTES
                     JXDATE     1   0  STARTING DATE
                     JXTIME         0  STARTING TIME

 33 KO          OUTPUT CONTROL VARIABLES
                     IPRNT          3  PRINT CONTROL
                     IPLOT          0  PLOT CONTROL
                     QSCAL         0.  HYDROGRAPH PLOT SCALE
                     IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                      IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                     ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                     ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                    TIMINT       .083  TIME INTERVAL IN HOURS


              SUBBASIN RUNOFF DATA

 23 BA          SUBBASIN CHARACTERISTICS
                     TAREA        .14  SUBBASIN AREA

                PRECIPITATION DATA

 24 PB               STORM       2.61  BASIN TOTAL PRECIPITATION

 26 PI            INCREMENTAL PRECIPITATION PATTERN
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
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                      .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                      .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                      .01      .01      .00      .00      .00      .00      .00      .00      .00      .00
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                      .00      .00      .00      .00      .00      .00      .00      .00

 31 LS          SCS LOSS RATE
                     STRTL        .27  INITIAL ABSTRACTION
                    CRVNBR      88.27  CURVE NUMBER
                     RTIMP        .00  PERCENT IMPERVIOUS AREA

 32 UD          SCS DIMENSIONLESS UNITGRAPH
                      TLAG        .31  LAG

                                                         ***

                                                   UNIT HYDROGRAPH
                                               21 END-OF-PERIOD ORDINATES
             25.       79.      154.      185.      174.      141.       93.       62.       43.       29.
             20.       13.        9.        6.        4.        3.        2.        1.        1.        0.
              0.

       ***            ***            ***            ***            ***

                        HYDROGRAPH AT STATION      SB13
                          FOR PLAN 1, RATIO =  .96

   TOTAL RAINFALL =    2.61, TOTAL LOSS =    1.11, TOTAL EXCESS =    1.50

  PEAK FLOW     TIME                      MAXIMUM AVERAGE FLOW
                                  6-HR        24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                        (CFS)
+     32.      13.17               14.          5.          5.           5.
                       (INCHES)   .987       1.482       1.482        1.482
                       (AC-FT)      7.         11.         11.          11.
```

CUMULATIVE AREA = 14 SQ MI

*** *** *** *** ***

HYDROGRAPH AT STATION SB13
FOR PLAN 1, RATIO = .96

TOTAL RAINFALL = 2.51, TOTAL LOSS = 1.10, TOTAL EXCESS = 1.41

	PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
				6-HR	24-HR	72-HR	23.92-HR
+	(CFS)	(HR)					
			(CFS)				
+	30.	13.17		13.	5.	5.	5.
			(INCHES)	.929	1.392	1.392	1.392
			(AC-FT)	7.	10.	10.	10.

CUMULATIVE AREA = .14 SQ MI

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*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

          **************
          *            *
  34 KK   *      HC6   *
          *            *
          **************

  36 KO        OUTPUT CONTROL VARIABLES
                    IPRNT          3  PRINT CONTROL
                    IPLOT          0  PLOT CONTROL
                    QSCAL         0.  HYDROGRAPH PLOT SCALE
                    IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                     IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                    ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                    ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                   TIMINT       .083  TIME INTERVAL IN HOURS


  35 HC        HYDROGRAPH COMBINATION
                    ICOMP          2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                                ***

     ***            ***            ***            ***            ***

                      HYDROGRAPH AT STATION       HC6
                        FOR PLAN 1, RATIO =   .96

  PEAK FLOW     TIME                              MAXIMUM AVERAGE FLOW
                                         6-HR        24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                              (CFS)
+       86.     13.42                    44.          16.         16.          16.
                            (INCHES)     .902        1.339       1.339        1.339
                             (AC-FT)     22.          32.         32.          32.

                       CUMULATIVE AREA =      .45 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

          **************
          *            *
  37 KK   *      SB9   *
          *            *
          **************

  40 IN        TIME DATA FOR INPUT TIME SERIES
                    JXMIN         30  TIME INTERVAL IN MINUTES
                   JXDATE      1   0  STARTING DATE
                   JXTIME          0  STARTING TIME

  48 KO        OUTPUT CONTROL VARIABLES
                    IPRNT          3  PRINT CONTROL
                    IPLOT          0  PLOT CONTROL
                    QSCAL         0.  HYDROGRAPH PLOT SCALE
                    IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                     IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                    ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                    ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                   TIMINT       .083  TIME INTERVAL IN HOURS


               SUBBASIN RUNOFF DATA

  38 BA        SUBBASIN CHARACTERISTICS
                    TAREA        .24  SUBBASIN AREA

               PRECIPITATION DATA

  39 PB             STORM       2.61  BASIN TOTAL PRECIPITATION

  41 PI          INCREMENTAL PRECIPITATION PATTERN
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
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                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
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.00	.00	.01	.01	.01	.01	.01	.01	.01	.01
.01	.01	.01	.01	.01	.01	.01	.01	.01	.01
.01	.01	.01	.01	.01	.01	.01	.01	.01	.01
.01	.01	.00	.00	.00	.00	.00	.00	.00	.00
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.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00			

46 LS SCS LOSS RATE

STRTL	.25	INITIAL ABSTRACTION
CRVNBR	89.00	CURVE NUMBER
RTIMP	.00	PERCENT IMPERVIOUS AREA

47 UD SCS DIMENSIONLESS UNITGRAPH

TLAG	.81	LAG

***

UNIT HYDROGRAPH
51 END-OF-PERIOD ORDINATES

4.	13.	25.	40.	61.	85.	107.	122.	131.	134.
133.	127.	118.	108.	95.	81.	67.	56.	47.	40.
35.	30.	26.	22.	18.	16.	13.	12.	10.	8.
7.	6.	5.	4.	4.	3.	3.	2.	2.	2.
1.	1.	1.	1.	1.	1.	1.	0.	0.	0.
0.									

*** *** *** *** ***

HYDROGRAPH AT STATION SB9
FOR PLAN 1, RATIO = .96

TOTAL RAINFALL = 2.61, TOTAL LOSS = 1.06, TOTAL EXCESS = 1.55

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	23.92-HR
+ (CFS)	(HR)					
		(CFS)				
+ 46.	13.58		26.	10.	10.	10.
		(INCHES)	1.013	1.513	1.513	1.513
		(AC-FT)	13.	19.	19.	19.

CUMULATIVE AREA = .24 SQ MI

*** *** *** *** ***

HYDROGRAPH AT STATION SB9
FOR PLAN 1, RATIO = .96

TOTAL RAINFALL = 2.51, TOTAL LOSS = 1.05, TOTAL EXCESS = 1.46

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	23.92-HR
+ (CFS)	(HR)					
		(CFS)				
+ 43.	13.67		24.	9.	9.	9.
		(INCHES)	.954	1.423	1.423	1.423
		(AC-FT)	12.	18.	18.	18.

CUMULATIVE AREA = .24 SQ MI

*** ***

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              **************
              *            *
49 KK         *     CV8    *
              *            *
              **************
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51 KO          OUTPUT CONTROL VARIABLES
                      IPRNT          3  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                       IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT       .083  TIME INTERVAL IN HOURS

             HYDROGRAPH ROUTING DATA

50 RD          MUSKINGUM-CUNGE CHANNEL ROUTING
                          L      4188.  CHANNEL LENGTH
                          S      .0025  SLOPE
                          N       .040  CHANNEL ROUGHNESS COEFFICIENT
                         CA        .00  CONTRIBUTING AREA
                      SHAPE       TRAP  CHANNEL SHAPE
                         WD       5.00  BOTTOM WIDTH OR DIAMETER
                          Z       4.00  SIDE SLOPE

                                                  ***
                                COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                     COMPUTATION TIME STEP
                ELEMENT   ALPHA      M        DT        DX       PEAK    TIME TO    VOLUME   MAXIMUM
                                                                          PEAK                CELERITY
                                            (MIN)      (FT)     (CFS)    (MIN)      (IN)      (FPS)

                  MAIN      .62     1.36      5.00    465.28     42.69    845.00     1.39      2.19

                                     INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                  MAIN      .62     1.36      5.00               42.69    845.00     1.39

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1793E+02 EXCESS= .0000E+00 OUTFLOW= .1752E+02 BASIN STORAGE= .4833E+00 PERCENT ERROR=   -.4


    ***          ***          ***          ***          ***

                       HYDROGRAPH AT STATION      CV8
                         FOR PLAN 1, RATIO =  .96

 PEAK FLOW     TIME                     MAXIMUM AVERAGE FLOW
                                  6-HR       24-HR       72-HR      23.92-HR
+   (CFS)      (HR)
                        (CFS)
+      43.     14.08               24.          9.          9.            9.
                      (INCHES)    .953       1.390       1.390         1.390
                       (AC-FT)     12.         17.         17.           17.

                      CUMULATIVE AREA =     .24 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

            **************
            *            *
52 KK       *    SB10    *
            *            *
            **************

55 IN          TIME DATA FOR INPUT TIME SERIES
                      JXMIN         30  TIME INTERVAL IN MINUTES
                     JXDATE      1   0  STARTING DATE
                     JXTIME          0  STARTING TIME

63 KO          OUTPUT CONTROL VARIABLES
                      IPRNT          3  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                       IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT       .083  TIME INTERVAL IN HOURS

             SUBBASIN RUNOFF DATA
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53 BA           SUBBASIN CHARACTERISTICS
                     TAREA         1.35  SUBBASIN AREA

                PRECIPITATION DATA

54 PB                STORM         2.61  BASIN TOTAL PRECIPITATION

56 PI             INCREMENTAL PRECIPITATION PATTERN
                     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
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                     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                     .00      .00      .01      .01      .01      .01      .01      .01      .01      .01
                     .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                     .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                     .01      .01      .00      .00      .00      .00      .00      .00      .00      .00
                     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
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                     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
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                     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                     .00      .00      .00      .00      .00      .00      .00

61 LS           SCS LOSS RATE
                     STRTL          .31  INITIAL ABSTRACTION
                    CRVNBR        86.42  CURVE NUMBER
                     RTIMP          .00  PERCENT IMPERVIOUS AREA

62 UD           SCS DIMENSIONLESS UNITGRAPH
                     TLAG           .46  LAG

                                                          ***

                                                     UNIT HYDROGRAPH
                                                30 END-OF-PERIOD ORDINATES
            99.      296.      606.      989.     1229.     1296.     1234.     1085.      888.      647.
           484.      367.      287.      219.      167.      127.       96.       73.       56.       43.
            33.       25.       19.       15.       12.        9.        7.        5.        2.        0.

     ***            ***            ***            ***            ***

                       HYDROGRAPH AT STATION     SB10
                         FOR PLAN 1, RATIO =  .96

   TOTAL RAINFALL =     2.61, TOTAL LOSS =     1.25, TOTAL EXCESS =     1.36

  PEAK FLOW      TIME                         MAXIMUM AVERAGE FLOW
                                     6-HR         24-HR        72-HR       23.92-HR
+    (CFS)       (HR)
                         (CFS)
+    271.       13.25                131.           49.          49.           49.
                       (INCHES)      .904         1.343        1.343         1.343
                        (AC-FT)       65.           96.          96.           96.

                       CUMULATIVE AREA =     1.35 SQ MI


     ***            ***            ***            ***            ***

                       HYDROGRAPH AT STATION     SB10
                         FOR PLAN 1, RATIO =  .96

   TOTAL RAINFALL =     2.51, TOTAL LOSS =     1.23, TOTAL EXCESS =     1.28

  PEAK FLOW      TIME                         MAXIMUM AVERAGE FLOW
                                     6-HR         24-HR        72-HR       23.92-HR
+    (CFS)       (HR)
                         (CFS)
+    254.       13.25                123.           46.          46.           46.
                       (INCHES)      .847         1.257        1.257         1.257
                        (AC-FT)       61.           90.          90.           90.
```

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                    CUMULATIVE AREA =     1.35 SQ MI

*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

            **************
            *            *
   64 KK    *      HC8   *
            *            *
            **************

   66 KO         OUTPUT CONTROL VARIABLES
                      IPRNT           3  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                       IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT        .083  TIME INTERVAL IN HOURS

   65 HC         HYDROGRAPH COMBINATION
                      ICOMP           2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                        ***

       ***          ***          ***          ***          ***

                     HYDROGRAPH AT STATION      HC8
                       FOR PLAN 1, RATIO =  .96

  PEAK FLOW     TIME                     MAXIMUM AVERAGE FLOW
                                   6-HR        24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                         (CFS)
+     282.    13.25                146.         54.         54.          54.
                       (INCHES)    .860       1.277       1.277        1.277
                        (AC-FT)     73.        108.        108.         108.

                    CUMULATIVE AREA =     1.58 SQ MI

*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

            **************
            *            *
   67 KK    *      CV1   *
            *            *
            **************

   69 KO         OUTPUT CONTROL VARIABLES
                      IPRNT           3  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                       IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT        .083  TIME INTERVAL IN HOURS

               HYDROGRAPH ROUTING DATA

   68 RD         MUSKINGUM-CUNGE CHANNEL ROUTING
                          L       2706.  CHANNEL LENGTH
                          S       .0025  SLOPE
                          N        .040  CHANNEL ROUGHNESS COEFFICIENT
                         CA         .00  CONTRIBUTING AREA
                      SHAPE        TRAP  CHANNEL SHAPE
                         WD        5.00  BOTTOM WIDTH OR DIAMETER
                          Z        4.00  SIDE SLOPE

                                                        ***
                              COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                   COMPUTATION TIME STEP
       ELEMENT    ALPHA       M        DT        DX       PEAK    TIME TO    VOLUME    MAXIMUM
                                                                    PEAK               CELERITY
                                     (MIN)      (FT)     (CFS)     (MIN)      (IN)      (FPS)

        MAIN       .62      1.36      5.00    901.95    278.04    810.00      1.27      3.61
```

```
                                INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                 MAIN          .62      1.36       5.00              278.04    810.00      1.27

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1078E+03 EXCESS= .0000E+00 OUTFLOW= .1068E+03 BASIN STORAGE= .1162E+01 PERCENT ERROR=   -.2


    ***            ***            ***            ***            ***

                   HYDROGRAPH AT STATION       CV1
                     FOR PLAN 1, RATIO =  .96

 PEAK FLOW      TIME                       MAXIMUM AVERAGE FLOW
                                   6-HR        24-HR       72-HR     23.92-HR
+   (CFS)       (HR)
                          (CFS)
+     278.     13.50                146.          54.          54.          54.
                        (INCHES)    .860       1.265       1.265        1.265
                         (AC-FT)     73.         107.        107.         107.

                       CUMULATIVE AREA =     1.58 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

            **************
            *            *
   70 KK    *      SB3   *
            *            *
            **************

   73 IN         TIME DATA FOR INPUT TIME SERIES
                      JXMIN          30  TIME INTERVAL IN MINUTES
                      JXDATE      1   0  STARTING DATE
                      JXTIME          0  STARTING TIME

   81 KO         OUTPUT CONTROL VARIABLES
                      IPRNT           3  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                      IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                      TIMINT       .083  TIME INTERVAL IN HOURS

               SUBBASIN RUNOFF DATA

   71 BA         SUBBASIN CHARACTERISTICS
                      TAREA         .13  SUBBASIN AREA

                 PRECIPITATION DATA

   72 PB              STORM        2.61  BASIN TOTAL PRECIPITATION

   74 PI           INCREMENTAL PRECIPITATION PATTERN
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
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                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .01       .01       .01       .01       .01       .01       .01       .01
                       .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                       .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                       .01       .01       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
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              .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
              .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
              .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
              .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
              .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
              .00      .00      .00      .00      .00      .00      .00

  79 LS        SCS LOSS RATE
                    STRTL          .25  INITIAL ABSTRACTION
                   CRVNBR        89.00  CURVE NUMBER
                    RTIMP          .00  PERCENT IMPERVIOUS AREA

  80 UD        SCS DIMENSIONLESS UNITGRAPH
                    TLAG           .44  LAG

                                                        ***

                                                  UNIT HYDROGRAPH
                                             28 END-OF-PERIOD ORDINATES
            10.       31.       65.      104.      124.      128.      118.      102.       79.       56.
            42.       32.       25.       18.       14.       11.        8.        6.        5.        3.
             3.        2.        1.        1.        1.        1.        0.        0.

      ***            ***            ***            ***            ***

                         HYDROGRAPH AT STATION      SB3
                           FOR PLAN 1, RATIO =  .96

    TOTAL RAINFALL =    2.61, TOTAL LOSS =    1.06, TOTAL EXCESS =    1.55

   PEAK FLOW     TIME                       MAXIMUM AVERAGE FLOW
                                  6-HR         24-HR        72-HR      23.92-HR
+    (CFS)       (HR)
                          (CFS)
+      29.      13.25                14.           5.           5.            5.
                       (INCHES)     1.019        1.531        1.531         1.531
                        (AC-FT)        7.          10.          10.           10.

                       CUMULATIVE AREA =     .13 SQ MI


      ***            ***            ***            ***            ***

                         HYDROGRAPH AT STATION      SB3
                           FOR PLAN 1, RATIO =  .96

    TOTAL RAINFALL =    2.51, TOTAL LOSS =    1.05, TOTAL EXCESS =    1.46

   PEAK FLOW     TIME                       MAXIMUM AVERAGE FLOW
                                  6-HR         24-HR        72-HR      23.92-HR
+    (CFS)       (HR)
                          (CFS)
+      28.      13.25                13.           5.           5.            5.
                       (INCHES)      .960        1.440        1.440         1.440
                        (AC-FT)        7.          10.          10.           10.

                       CUMULATIVE AREA =     .13 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

           **************
           *            *
  82 KK    *       HC3  *
           *            *
           **************

  84 KO        OUTPUT CONTROL VARIABLES
                    IPRNT            3  PRINT CONTROL
                    IPLOT            0  PLOT CONTROL
                    QSCAL           0.  HYDROGRAPH PLOT SCALE
                    IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                     IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                    ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                    ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                   TIMINT         .083  TIME INTERVAL IN HOURS

  83 HC        HYDROGRAPH COMBINATION
                    ICOMP            2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                        ***

      ***            ***            ***            ***            ***
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                    HYDROGRAPH AT STATION      HC3
                      FOR PLAN 1, RATIO =  .96

 ..EAK FLOW     TIME                          MAXIMUM AVERAGE FLOW
                                        6-HR        24-HR        72-HR      23.92-HR
+   (CFS)       (HR)
                              (CFS)
+     303.      13.42                   159.          59.          59.           59.
                            (INCHES)    .867        1.278        1.278        1.278
                            (AC-FT)      79.         117.         117.         117.

                          CUMULATIVE AREA =    1.71 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


              **************
              *            *
  85 KK       *      CV2   *
              *            *
              **************

  87 KO             OUTPUT CONTROL VARIABLES
                          IPRNT           3  PRINT CONTROL
                          IPLOT           0  PLOT CONTROL
                          QSCAL          0.  HYDROGRAPH PLOT SCALE
                          IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                           IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                          ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                          ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                         TIMINT        .083  TIME INTERVAL IN HOURS


                 HYDROGRAPH ROUTING DATA

  86 RD             MUSKINGUM-CUNGE CHANNEL ROUTING
                              L       2294.  CHANNEL LENGTH
                              S       .0025  SLOPE
                              N        .040  CHANNEL ROUGHNESS COEFFICIENT
                             CA         .00  CONTRIBUTING AREA
                          SHAPE        TRAP  CHANNEL SHAPE
                             WD        5.00  BOTTOM WIDTH OR DIAMETER
                              Z        4.00  SIDE SLOPE

                                                                      ***
                                      COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                             COMPUTATION TIME STEP
                       ELEMENT    ALPHA       M         DT         DX        PEAK     TIME TO      VOLUME     MAXIMUM
                                                                                        PEAK                  CELERITY
                                                      (MIN)       (FT)      (CFS)     (MIN)        (IN)       (FPS)

                         MAIN        .62      1.36       5.00     764.80    300.47    815.00       1.27        3.68

                                             INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL


                         MAIN        .62      1.36       5.00               300.47    815.00       1.27

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1167E+03 EXCESS= .0000E+00 OUTFLOW= .1157E+03 BASIN STORAGE= .1059E+01 PERCENT ERROR=    -.1


      ***            ***            ***            ***            ***

                    HYDROGRAPH AT STATION      CV2
                      FOR PLAN 1, RATIO =  .96

  PEAK FLOW     TIME                          MAXIMUM AVERAGE FLOW
                                        6-HR        24-HR        72-HR      23.92-HR
+   (CFS)       (HR)
                              (CFS)
+     300.      13.58                   159.          58.          58.           58.
                            (INCHES)    .866        1.268        1.268        1.268
                            (AC-FT)      79.         116.         116.         116.

                          CUMULATIVE AREA =    1.71 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***
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              **************
              *            *
  1 KK        *     SB2    *
              *            *
              **************

 91 IN             TIME DATA FOR INPUT TIME SERIES
                        JXMIN        30  TIME INTERVAL IN MINUTES
                        JXDATE     1   0  STARTING DATE
                        JXTIME        0  STARTING TIME

 99 KO             OUTPUT CONTROL VARIABLES
                        IPRNT         3  PRINT CONTROL
                        IPLOT         0  PLOT CONTROL
                        QSCAL        0.  HYDROGRAPH PLOT SCALE
                        IPNCH         0  PUNCH COMPUTED HYDROGRAPH
                         IOUT        22  SAVE HYDROGRAPH ON THIS UNIT
                        ISAV1         1  FIRST ORDINATE PUNCHED OR SAVED
                        ISAV2       288  LAST ORDINATE PUNCHED OR SAVED
                       TIMINT      .083  TIME INTERVAL IN HOURS


                 SUBBASIN RUNOFF DATA

 89 BA             SUBBASIN CHARACTERISTICS
                        TAREA       .38  SUBBASIN AREA

                   PRECIPITATION DATA

 90 PB                  STORM      2.61  BASIN TOTAL PRECIPITATION

 92 PI               INCREMENTAL PRECIPITATION PATTERN
                         .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                         .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                         .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                         .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                         .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                         .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                         .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
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                         .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                         .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                         .01      .01      .00      .00      .00      .00      .00      .00      .00      .00
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                         .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                         .00      .00      .00      .00      .00      .00      .00

 97 LS             SCS LOSS RATE
                        STRTL       .28  INITIAL ABSTRACTION
                       CRVNBR     87.70  CURVE NUMBER
                        RTIMP       .00  PERCENT IMPERVIOUS AREA

 98 UD             SCS DIMENSIONLESS UNITGRAPH
                         TLAG       .70  LAG

                                                       ***

                                                UNIT HYDROGRAPH
                                           44 END-OF-PERIOD ORDINATES
            10.       30.       58.       96.      145.      192.      225.      244.      246.      240.
           223.      202.      177.      146.      117.       96.       80.       67.       57.       47.
            39.       33.       27.       23.       19.       16.       13.       11.        9.        8.
             6.        5.        4.        4.        3.        3.        2.        2.        2.        1.
             1.        1.        0.        0.

   ***            ***            ***            ***            ***

                      HYDROGRAPH AT STATION      SB2
                        FOR PLAN 1, RATIO =  .96

  TOTAL RAINFALL =     2.61, TOTAL LOSS =     1.16, TOTAL EXCESS =     1.45
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 PEAK FLOW     TIME                              MAXIMUM AVERAGE FLOW
                                       6-HR        24-HR       72-HR      23.92-HR
   (CFS)       (HR)
                          (CFS)
+    72.      13.50                     39.          15.         15.          15.
                        (INCHES)       .956        1.422       1.422        1.422
                         (AC-FT)        19.          29.         29.          29.

                        CUMULATIVE AREA =     .38 SQ MI


     ***                 ***            ***           ***                    ***

                          HYDROGRAPH AT STATION      SB2
                            FOR PLAN 1, RATIO =   .96

    TOTAL RAINFALL =    2.51, TOTAL LOSS =    1.14, TOTAL EXCESS =     1.36

 PEAK FLOW     TIME                              MAXIMUM AVERAGE FLOW
                                       6-HR        24-HR       72-HR      23.92-HR
+  (CFS)       (HR)
                          (CFS)
+    68.      13.50                     37.          14.         14.          14.
                        (INCHES)       .899        1.335       1.335        1.335
                         (AC-FT)        18.          27.         27.          27.

                        CUMULATIVE AREA =     .38 SQ MI
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*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

             **************
             *            *
 100 KK      *      HC2   *
             *            *
             **************

 102 KO             OUTPUT CONTROL VARIABLES
                          IPRNT            3  PRINT CONTROL
                          IPLOT            0  PLOT CONTROL
                          QSCAL           0.  HYDROGRAPH PLOT SCALE
                          IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                           IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                          ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                          ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                         TIMINT         .083  TIME INTERVAL IN HOURS


 101 HC             HYDROGRAPH COMBINATION
                          ICOMP            2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                                     ***

      ***            ***            ***            ***            ***

                            HYDROGRAPH AT STATION      HC2
                              FOR PLAN 1, RATIO =  .96

 PEAK FLOW      TIME                        MAXIMUM AVERAGE FLOW
                                    6-HR        24-HR       72-HR     23.92-HR
+   (CFS)       (HR)
                          (CFS)
+     368.     13.58                196.          72.         72.          72.
                        (INCHES)    .871        1.280       1.280        1.280
                         (AC-FT)     97.         142.        142.         142.

                         CUMULATIVE AREA =     2.09 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

             **************
             *            *
 103 KK      *      CV3   *
             *            *
             **************

 105 KO             OUTPUT CONTROL VARIABLES
                          IPRNT            3  PRINT CONTROL
                          IPLOT            0  PLOT CONTROL
                          QSCAL           0.  HYDROGRAPH PLOT SCALE
                          IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                           IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                          ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                          ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                         TIMINT         .083  TIME INTERVAL IN HOURS

                  HYDROGRAPH ROUTING DATA

 104 RD             MUSKINGUM-CUNGE CHANNEL ROUTING
                              L        2868.  CHANNEL LENGTH
                              S        .0025  SLOPE
                              N         .040  CHANNEL ROUGHNESS COEFFICIENT
                             CA          .00  CONTRIBUTING AREA
                          SHAPE         TRAP  CHANNEL SHAPE
                             WD         5.00  BOTTOM WIDTH OR DIAMETER
                              Z         4.00  SIDE SLOPE

                                                                     ***
                                   COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                         COMPUTATION TIME STEP
                ELEMENT    ALPHA       M         DT        DX       PEAK    TIME TO    VOLUME    MAXIMUM
                                                                               PEAK               CELERITY
                                                (MIN)      (FT)     (CFS)     (MIN)      (IN)      (FPS)

                 MAIN       .62      1.36      5.00    955.92    363.86    825.00      1.27       3.88

                                        INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL
```

```
                    MAIN         .62       1.36       5.00              363.86    825.00      1.27

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1426E+03 EXCESS= .0000E+00 OUTFLOW= .1413E+03 BASIN STORAGE= .1567E+01 PERCENT ERROR=   -.2


     ***            ***            ***            ***            ***

                       HYDROGRAPH AT STATION      CV3
                         FOR PLAN 1, RATIO =  .96

  PEAK FLOW     TIME                            MAXIMUM AVERAGE FLOW
                                        6-HR        24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                            (CFS)
+    364.      13.75                    196.          71.         71.          71.
                          (INCHES)      .871        1.268       1.268        1.268
                           (AC-FT)       97.         141.        141.         141.

                         CUMULATIVE AREA =     2.09 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


           **************
           *            *
 106 KK    *      SB8   *
           *            *
           **************

 109 IN          TIME DATA FOR INPUT TIME SERIES
                      JXMIN          30  TIME INTERVAL IN MINUTES
                     JXDATE       1   0  STARTING DATE
                     JXTIME           0  STARTING TIME

 117 KO          OUTPUT CONTROL VARIABLES
                      IPRNT           3  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                       IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT        .083  TIME INTERVAL IN HOURS


               SUBBASIN RUNOFF DATA

 107 BA          SUBBASIN CHARACTERISTICS
                      TAREA         .12  SUBBASIN AREA

                 PRECIPITATION DATA

 108 PB               STORM        2.61  BASIN TOTAL PRECIPITATION

 110 PI             INCREMENTAL PRECIPITATION PATTERN
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .01      .01      .01      .01      .01      .01      .01      .01
                      .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                      .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                      .01      .01      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
```

```
                 .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                 .00       .00       .00       .00       .00       .00       .00

 .15 LS          SCS LOSS RATE
                      STRTL          .42  INITIAL ABSTRACTION
                     CRVNBR        82.79  CURVE NUMBER
                      RTIMP          .00  PERCENT IMPERVIOUS AREA

 116 UD          SCS DIMENSIONLESS UNITGRAPH
                       TLAG          .50  LAG

                                                                ***

                                                           UNIT HYDROGRAPH
                                                      32 END-OF-PERIOD ORDINATES
              7.        22.        45.        75.        98.       109.       109.       100.        87.        70.
             51.        40.        31.        25         19.        15.        11.         9.         7.         5.
              4.         3.         3.         2.         2.         1.         1.         1.         1.         0.
              0.         0.

      ***             ***             ***             ***             ***

                           HYDROGRAPH AT STATION       SB8
                             FOR PLAN 1, RATIO =  .96

    TOTAL RAINFALL =    2.61, TOTAL LOSS =    1.48, TOTAL EXCESS =    1.13

  PEAK FLOW      TIME                       MAXIMUM AVERAGE FLOW
                                     6-HR        24-HR        72-HR      23.92-HR
 +   (CFS)       (HR)
                           (CFS)
 +      20.     13.33                  10.           4.           4.            4.
                         (INCHES)     .752        1.107        1.107         1.107
                         (AC-FT)        5.           7.           7.            7.

                         CUMULATIVE AREA =      .12 SQ MI


      ***             ***             ***             ***             ***

                           HYDROGRAPH AT STATION       SB8
                             FOR PLAN 1, RATIO =  .96

    TOTAL RAINFALL =    2.51, TOTAL LOSS =    1.46, TOTAL EXCESS =    1.05

  :AK FLOW       TIME                       MAXIMUM AVERAGE FLOW
                                     6-HR        24-HR        72-HR      23.92-HR
 +   (CFS)       (HR)
                           (CFS)
 +      19.     13.33                   9.           3.           3.            3.
                         (INCHES)     .700        1.029        1.029         1.029
                         (AC-FT)        5.           7.           7.            7.

                         CUMULATIVE AREA =      .12 SQ MI



*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


             **************
             *            *
 118 KK      *      HC1   *
             *            *
             **************

 120 KO          OUTPUT CONTROL VARIABLES
                      IPRNT            3  PRINT CONTROL
                      IPLOT            0  PLOT CONTROL
                      QSCAL           0.  HYDROGRAPH PLOT SCALE
                      IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                       IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT         .083  TIME INTERVAL IN HOURS

 119 HC          HYDROGRAPH COMBINATION
                      ICOMP            2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                                ***

      ***             ***             ***             ***             ***

                           HYDROGRAPH AT STATION       HC1
                             FOR PLAN 1, RATIO =   96
```

```
 PEAK FLOW     TIME                           MAXIMUM AVERAGE FLOW
                                   6-HR        24-HR       72-HR      23.92-HR
   (CFS)        (HR)
                        (CFS)
+    379.      13.75                205.         75.         75.          75.
                      (INCHES)      .861       1.255       1.255        1.255
                       (AC-FT)      102.         148        148.         148.

                       CUMULATIVE AREA =    2.21 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
 121 KK   *      CV4   *
          *            *
          **************

 123 KO        OUTPUT CONTROL VARIABLES
                     IPRNT          3  PRINT CONTROL
                     IPLOT          0  PLOT CONTROL
                     QSCAL         0.  HYDROGRAPH PLOT SCALE
                     IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                      IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                     ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                     ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                    TIMINT       .083  TIME INTERVAL IN HOURS


             HYDROGRAPH ROUTING DATA

 122 RD        MUSKINGUM-CUNGE CHANNEL ROUTING
                         L      2880.  CHANNEL LENGTH
                         S      .0025  SLOPE
                         N       .040  CHANNEL ROUGHNESS COEFFICIENT
                        CA        .00  CONTRIBUTING AREA
                     SHAPE       TRAP  CHANNEL SHAPE
                        WD       5.00  BOTTOM WIDTH OR DIAMETER
                         Z       4.00  SIDE SLOPE

                                                        ***
                                COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                       COMPUTATION TIME STEP
                 ELEMENT    ALPHA       M         DT        DX        PEAK    TIME TO     VOLUME    MAXIMUM
                                                                               PEAK                 CELERITY
                                                 (MIN)     (FT)      (CFS)    (MIN)       (IN)       (FPS)

                  MAIN        .62      1.36      5.00     960.13    375.46    835.00      1.24       3.91

                                       INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL


                  MAIN        .62      1.36      5.00               375.46    835.00      1.24

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1481E+03 EXCESS= .0000E+00 OUTFLOW= .1466E+03 BASIN STORAGE= .1689E+01 PERCENT ERROR=   -.2


     ***             ***             ***             ***             ***

                         HYDROGRAPH AT STATION      CV4
                           FOR PLAN 1, RATIO =  .96

 PEAK FLOW     TIME                           MAXIMUM AVERAGE FLOW
                                   6-HR        24-HR       72-HR      23.92-HR
+  (CFS)        (HR)
                        (CFS)
+    375.      13.92                205.         74.         74.          74.
                      (INCHES)      .861       1.243       1.243        1.243
                       (AC-FT)      101.        146.        146.         146.

                       CUMULATIVE AREA =    2.21 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
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124 KK      *      SB12  *
            *            *
            **************

127 IN          TIME DATA FOR INPUT TIME SERIES
                     JXMIN          30  TIME INTERVAL IN MINUTES
                    JXDATE      1    0  STARTING DATE
                    JXTIME           0  STARTING TIME

135 KO          OUTPUT CONTROL VARIABLES
                     IPRNT           3  PRINT CONTROL
                     IPLOT           0  PLOT CONTROL
                     QSCAL          0.  HYDROGRAPH PLOT SCALE
                     IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                      IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                     ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                     ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                    TIMINT        .083  TIME INTERVAL IN HOURS


              SUBBASIN RUNOFF DATA

125 BA          SUBBASIN CHARACTERISTICS
                     TAREA        1.19  SUBBASIN AREA

                PRECIPITATION DATA

126 PB               STORM        2.61  BASIN TOTAL PRECIPITATION

128 PI            INCREMENTAL PRECIPITATION PATTERN
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .01       .01       .01       .01       .01       .01       .01       .01
                      .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                      .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                      .01       .01       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                      .00       .00       .00       .00       .00       .00       .00

133 LS          SCS LOSS RATE
                    STRTL         .27  INITIAL ABSTRACTION
                   CRVNBR       88.24  CURVE NUMBER
                    RTIMP         .00  PERCENT IMPERVIOUS AREA

134 UD          SCS DIMENSIONLESS UNITGRAPH
                     TLAG         .49  LAG

                                                         ***

                                                     UNIT HYDROGRAPH
                                                31 END-OF-PERIOD ORDINATES
             76.       224.       458.       763.       969.      1078.      1074.       968.       834.       650.
            481.       370.       289.       227.       175.       136.       106.        81.        62.        49.
             38.        29.        23.        18.        14.        11          9.         7.         5.         3.
              2.

      ***             ***             ***             ***             ***

                        HYDROGRAPH AT STATION      SB12
                          FOR PLAN 1, RATIO =   .96

   TOTAL RAINFALL =     2.61, TOTAL LOSS =     1.12, TOTAL EXCESS =     1.49

PEAK FLOW       TIME                          MAXIMUM AVERAGE FLOW
                                   6-HR          24-HR         72-HR       23.92-HR
+   CFS)        (HR)
```

```
                                  (CFS)
    257.       13.33                           126.          47.          47.           47.
                               (INCHES)        .964        1.471        1.471        1.471
                                (AC-FT)         63.          94.          94.          94.

                               CUMULATIVE AREA =     1.19 SQ MI


      ***               ***              ***              ***              ***

                                HYDROGRAPH AT STATION      SB12
                                  FOR PLAN 1, RATIO =   .96

     TOTAL RAINFALL =     2.51, TOTAL LOSS =    1.10, TOTAL EXCESS =     1.40

   PEAK FLOW      TIME                            MAXIMUM AVERAGE FLOW
                                                6-HR        24-HR        72-HR      23.92-HR
 +   (CFS)        (HR)
                                  (CFS)
 +     242.       13.33                          119.          45.          45.          45.
                               (INCHES)          .926        1.382        1.382        1.382
                                (AC-FT)           59.          88.          88.          88.

                               CUMULATIVE AREA =     1.19 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

              **************
              *            *
   136 KK     *      HC7   *
              *            *
              **************

   138 KO           OUTPUT CONTROL VARIABLES
                          IPRNT            3  PRINT CONTROL
                          IPLOT            0  PLOT CONTROL
                          QSCAL           0.  HYDROGRAPH PLOT SCALE
                          IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                           IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                          ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                          ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                         TIMINT         .083  TIME INTERVAL IN HOURS


   137 HC           HYDROGRAPH COMBINATION
                          ICOMP            2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                                      ***

      ***               ***              ***              ***              ***

                                HYDROGRAPH AT STATION       HC7
                                  FOR PLAN 1, RATIO =   .96

   PEAK FLOW      TIME                            MAXIMUM AVERAGE FLOW
                                                6-HR        24-HR        72-HR      23.92-HR
 +   (CFS)        (HR)
                                  (CFS)
 +     551.       13.83                          320.         119.         119.         119.
                               (INCHES)          .875        1.292        1.292        1.292
                                (AC-FT)          159.         234.         234.         234.

                               CUMULATIVE AREA =     3.40 SQ MI

1

                 PEAK FLOW AND STAGE (END-OF-PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
                                  FLOWS IN CUBIC FEET PER SECOND,  AREA IN SQUARE MILES
                                                 TIME TO PEAK IN HOURS


                                                         RATIOS APPLIED TO PRECIPITATION
  OPERATION        STATION     AREA    PLAN              RATIO 1
                                                            .96

  HYDROGRAPH AT
 +                    SB11        .32      1    FLOW         63.
                                                TIME       13.25

  ROUTED TO
 +                     CV7        .32      1    FLOW         62.
                                                TIME       13.50
```

```
HYDROGRAPH AT
                  SB13      .14      1   FLOW         30.
                                         TIME      13.17

  2 COMBINED AT
+                 HC6       .45      1   FLOW         86.
                                         TIME      13.42

 HYDROGRAPH AT
+                 SB9       .24      1   FLOW         43.
                                         TIME      13.67

 ROUTED TO
+                 CV8       .24      1   FLOW         43.
                                         TIME      14.08

 HYDROGRAPH AT
+                 SB10     1.35      1   FLOW        254.
                                         TIME      13.25

  2 COMBINED AT
+                 HC8      1.58      1   FLOW        282.
                                         TIME      13.25

 ROUTED TO
+                 CV1      1.58      1   FLOW        278.
                                         TIME      13.50

 HYDROGRAPH AT
+                 SB3       .13      1   FLOW         28.
                                         TIME      13.25

  2 COMBINED AT
+                 HC3      1.71      1   FLOW        303.
                                         TIME      13.42

 ROUTED TO
+                 CV2      1.71      1   FLOW        300.
                                         TIME      13.58

 HYDROGRAPH AT
+                 SB2       .38      1   FLOW         68.
                                         TIME      13.50

  ` COMBINED AT
                  HC2      2.09      1   FLOW        368.
                                         TIME      13.58

 ROUTED TO
+                 CV3      2.09      1   FLOW        364.
                                         TIME      13.75

 HYDROGRAPH AT
+                 SB8       .12      1   FLOW         19.
                                         TIME      13.33

  2 COMBINED AT
+                 HC1      2.21      1   FLOW        379.
                                         TIME      13.75

 ROUTED TO
+                 CV4      2.21      1   FLOW        375.
                                         TIME      13.92

 HYDROGRAPH AT
+                 SB12     1.19      1   FLOW        242.
                                         TIME      13.33

  2 COMBINED AT
+                 HC7      3.40      1   FLOW        551.
                                         TIME      13.83
1
                                         SUMMARY OF KINEMATIC WAVE - MUSKINGUM-CUNGE ROUTING
                                             (FLOW IS DIRECT RUNOFF WITHOUT BASE FLOW)
                                                                             INTERPOLATED TO
                                                                          COMPUTATION INTERVAL
         ISTAQ    ELEMENT     DT       PEAK   TIME TO    VOLUME      DT       PEAK    TIME TO     VOLUME
                                                PEAK                                    PEAK

                             (MIN)     (CFS)   (MIN)      (IN)     (MIN)     (CFS)     (MIN)       (IN)

        FOR PLAN = 1  RATIO=   .00
              CV7  MANE       5.00     62.09   810.00     1.32      5.00     62.09    810.00       1.32

CONTINUITY SUMMARY (AC-FT) - INFLOW= .2266E+02 EXCESS= .0000E+00 OUTFLOW= .2242E+02 BASIN STORAGE= .2885E+00 PERCENT ERROR=   -.2
```

```
          FOR PLAN = 1  RATIO=   .00
               CV5  MANE          5.00     42.69    845.00     1.39     5.00     42.69    845.00      1.39

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1793E+02 EXCESS= .0000E+00 OUTFLOW= .1752E+02 BASIN STORAGE= .4833E+00 PERCENT ERROR=   -.4

          FOR PLAN = 1  RATIO=   .00
               CV1  MANE          5.00    278.04    810.00     1.27     5.00    278.04    810.00      1.27

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1078E+03 EXCESS= .0000E+00 OUTFLOW= .1068E+03 BASIN STORAGE= .1162E+01 PERCENT ERROR=   -.2

          FOR PLAN = 1  RATIO=   .00
               CV2  MANE          5.00    300.47    815.00     1.27     5.00    300.47    815.00      1.27

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1167E+03 EXCESS= .0000E+00 OUTFLOW= .1157E+03 BASIN STORAGE= .1059E+01 PERCENT ERROR=   -.1

          FOR PLAN = 1  RATIO=   .00
               CV3  MANE          5.00    363.86    825.00     1.27     5.00    363.86    825.00      1.27

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1426E+03 EXCESS= .0000E+00 OUTFLOW= .1413E+03 BASIN STORAGE= .1567E+01 PERCENT ERROR=   -.2

          FOR PLAN = 1  RATIO=   .00
               CV4  MANE          5.00    375.46    835.00     1.24     5.00    375.46    835.00      1.24

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1481E+03 EXCESS= .0000E+00 OUTFLOW= .1466E+03 BASIN STORAGE= .1689E+01 PERCENT ERROR=   -.2


*** NORMAL END OF HEC-1 ***
```

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.

2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.

3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table 2-2b Runoff curve numbers for cultivated agricultural lands [1]

Cover description			Curve numbers for hydrologic soil group			
Cover type	Treatment [2]	Hydrologic condition [3]	A	B	C	D
Fallow	Bare soil	—	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T+ CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T+ CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

1 Average runoff condition, and I_a=0.2S
2 Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.
3 Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥ 20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Table 2-2c Runoff curve numbers for other agricultural lands [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. [2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. [3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4]	48	65	73
Woods—grass combination (orchard or tree farm). [5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. [6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 [4]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

1 Average runoff condition, and $I_a = 0.2S$.

2 *Poor:* <50%) ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

3 *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

4 Actual curve number is less than 30; use CN = 30 for runoff computations.

5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

6 *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Table 2-2d Runoff curve numbers for arid and semiarid rangelands [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition [2]	A [3]	B	C	D
Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element.	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush.	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper—pinyon, juniper, or both; grass understory.	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory.	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus.	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

1 Average runoff condition, and I_a, = 0.2S. For range in humid regions, use table 2-2c.
2 Poor: <30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: > 70% ground cover.
3 Curve numbers for group A have been developed only for desert shrub.

Wasatch Regional Area Weighted Curve Numbers

SB10

	Area (ac)	Soil Type	CN	Weighted CN
	0.877	B	71	0.07
	119.535	B	71	10.09
	720.723	D	89	76.26
Total	841.135			86.42

SB11

	Area (ac)		CN	Weighted CN
	16.106	B	71	5.90
	177.717	D	89	81.60
Total	193.823			87.50

SB2

	Area (ac)		CN	Weighted CN
	16.775	B	71	5.14
	214.842	D	89	82.55
Total	231.617			87.70

SB3

	Area (ac)		CN	Weighted CN
	58.775	D	89	89.00
Total	58.775			89.00

SB8

	Area (ac)		CN	Weighted CN
	49.034	D	89	58.32
	25.800	B	71	24.48
Total	74.834			82.79

SB9

	Area (ac)		CN	Weighted CN
	141.574	D	89	89.00
Total	141.574			89.00

SB13

from this site. A DOQ is a computer-generated image of an aerial photograph in which image displacement caused by terrain relief and camera tilts has been removed. It combines the image characteristics of a photograph with the geometric qualities of a map. Visit the USGS for more information.

atershed/Stream Flow Information -

Find the Watershed for this location using the U.S. Environmental Protection Agency's site.

Climate Data Sources -

Precipitation frequency results are based on data from a variety of sources, but largely NCDC. The following links provide general information
about observing sites in the area, regardless of if their data was used in this study. For detailed information about the stations used in this study,
please refer to our documentation.

Using the **National Climatic Data Center's (NCDC)** station search engine, locate other climate stations within:

+/-30 minutes ...OR... +/-1 degree of this location (40.85234/-112.77226). Digital ASCII data can be obtained directly from NCDC.

Find Natural Resources Conservation Service (NRCS) SNOTEL (SNOwpack TELemetry) stations by visiting the **Western Regional Climate Center's state-specific SNOTEL station maps**.

Hydrometeorological Design Studies Center
DOC/NOAA/National Weather Service
1325 East-West Highway
Silver Spring, MD 20910

1) 713-1669
Questions?: HDSC.Questions@noaa.gov

Disclaimer

Chapter 3 Time of Concentration and Travel Time

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge. But T_c can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading.

Factors affecting time of concentration and travel time

Surface roughness

One of the most significant effects of urban development on flow velocity is less retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by urban development: the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

Channel shape and flow patterns

In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

Slope

Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

Computation of travel time and time of concentration

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time (T_t) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600V} \qquad \text{[eq. 3-1]}$$

where:

T_t = travel time (hr)
L = flow length (ft)
V = average velocity (ft/s)
3600 = conversion factor from seconds to hours.

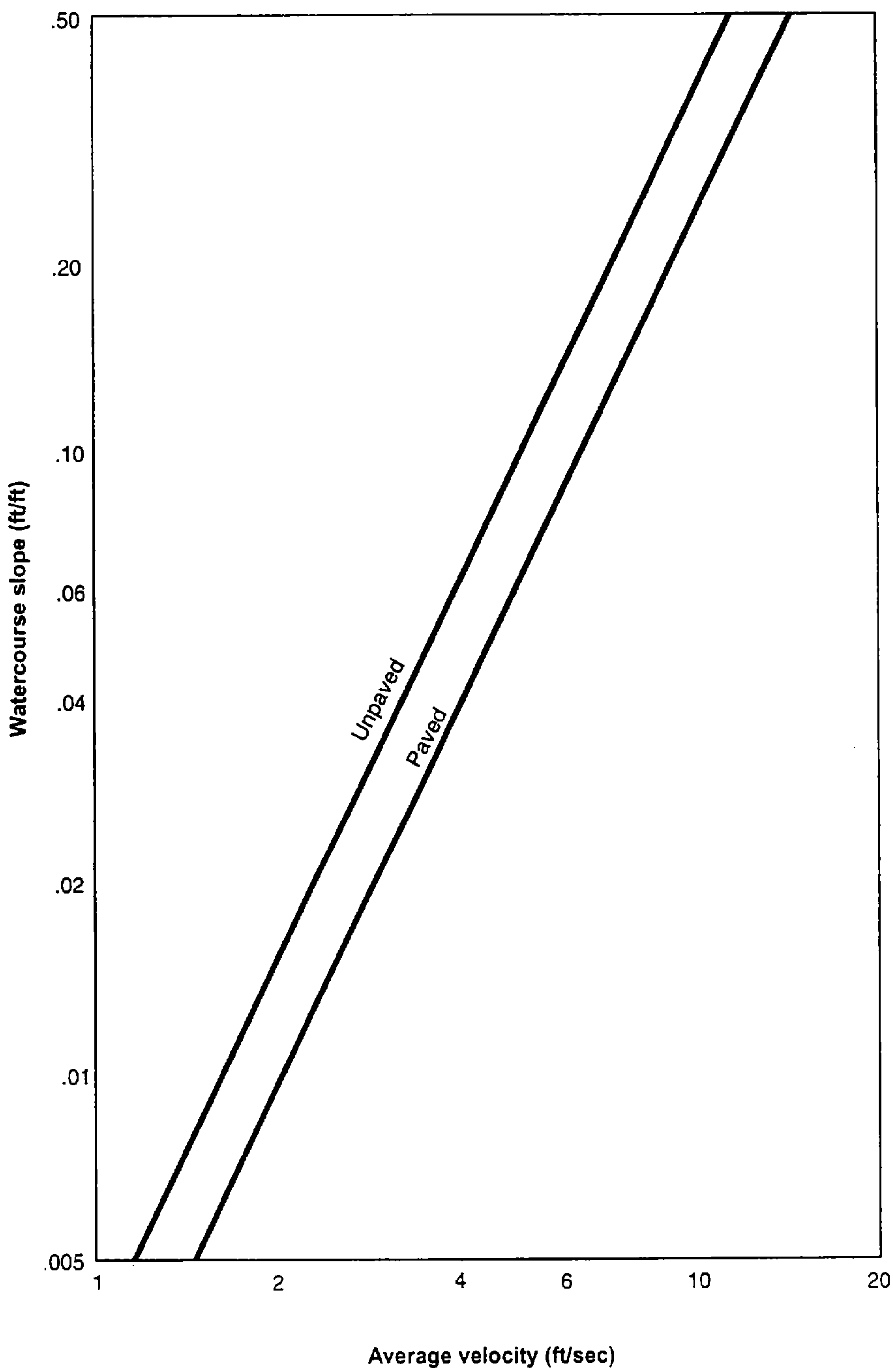
Time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t_1} + T_{t_2} + \ldots T_{t_m} \qquad \text{[eq. 3-2]}$$

where:

T_c = time of concentration (hr)
m = number of flow segments

Figure 3-1 Average velocities for estimating travel time for shallow concentrated flow



Sheet flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 3-1 gives Manning's n values for sheet flow for various surface conditions.

Table 3-1 Roughness coefficients (Manning's n) for sheet flow

Surface description	n [1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods:[3]	
Light underbrush	0.40
Dense underbrush	0.80

1 The n values are a composite of information compiled by Engman (1986).
2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
3 When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overtop and Meadows 1976) to compute T_t:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}\, s^{0.4}} \qquad \text{[eq. 3-3]}$$

where:

T_t = travel time (hr),
n = Manning's roughness coefficient (table 3-1)
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (land slope, ft/ft)

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

Shallow concentrated flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bank-full elevation.

Manning's equation is:

$$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \qquad \text{[eq. 3-4]}$$

where:

- V = average velocity (ft/s)
- r = hydraulic radius (ft) and is equal to a/p_w
  - a = cross sectional flow area (ft²)
  - p_w = wetted perimeter (ft)
- s = slope of the hydraulic grade line (channel slope, ft/ft)
- n = Manning's roughness coefficient for open channel flow.

Manning's n values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using equation 3-4, T_t for the channel segment can be estimated using equation 3-1.

Reservoirs or lakes

Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero.

Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation 3-3 was developed for use with the four standard rainfall intensity-duration relationships.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.
- The minimum T_c used in TR-55 is 0.1 hour.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. The procedures in TR-55 can be used to determine the peak flow upstream of the culvert. Detailed storage routing procedures should be used to determine the outflow through the culvert.

Example 3-1

The sketch below shows a watershed in Dyer County, northwestern Tennessee. The problem is to compute T_c at the outlet of the watershed (point D). The 2-year 24-hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute T_c, first determine T_t for each segment from the following information:

Segment AB: Sheet flow; dense grass; slope (s) = 0.01 ft/ft; and length (L) = 100 ft. Segment BC: Shallow concentrated flow; unpaved; s = 0.01 ft/ft; and L = 1,400 ft. Segment CD: Channel flow; Manning's n = .05; flow area (a) = 27 ft²; wetted perimeter (p_w) = 28.2 ft; s = 0.005 ft/ft; and L = 7,300 ft.

See figure 3-2 for the computations made on worksheet 3.

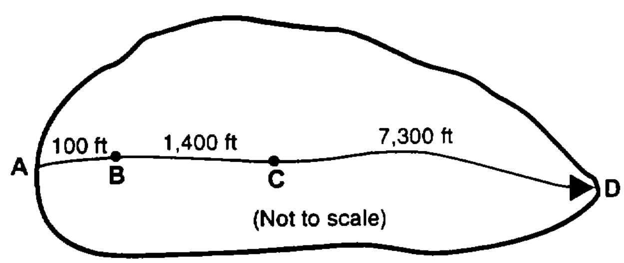


Figure 3-2 Worksheet 3 for example 3-1

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project: Heavenly Acres	By: DW	Date: 10/6/85
Location: Dyer County, Tennessee	Checked: NM	Date: 10/8/85

Check one: ☐ Present ☒ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

	Segment ID		AB		
1.	Surface description (table 3-1)		Dense Grass		
2.	Manning's roughness coefficient, n (table 3-1)		0.24		
3.	Flow length, L (total L ≤ 300 ft)	ft	100		
4.	Two-year 24-hour rainfall, P_2	in	3.6		
5.	Land slope, s	ft/ft	0.01		
6.	$T_t = \dfrac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t	hr	0.30	+	= 0.30

	Segment ID		BC		
7.	Surface description (paved or unpaved)		Unpaved		
8.	Flow length, L	ft	1400		
9.	Watercourse slope, s	ft/ft	0.01		
10.	Average velocity, V (figure 3-1)	ft/s	1.6		
11.	$T_t = \dfrac{L}{3600\ V}$ Compute T_t	hr	0.24	+	= 0.24

	Segement ID		CD		
12.	Cross sectional flow area, a	ft^2	27		
13.	Wetted perimeter, p_w	ft	28.2		
14.	Hydraulic radius, $r = \dfrac{a}{p_w}$ Compute r	ft	0.957		
15	Channel slope, s	ft/ft	0.005		
16.	Manning's roughness coefficient, n		0.05		
17.	$V = \dfrac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V	ft/s	2.05		
18.	Flow length, L	ft	7300		
19.	$T_t = \dfrac{L}{3600\ V}$ Compute T_t	hr	0.99	+	= 0.99
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	Hr			1.53

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project	By	Date
Allied Waste- Wasatch Regional	Gordon Jones	11/4/04
Location	**Checked**	**Date**
SB10		

Check one: ☒ Present ☐ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Segment ID				
1. Surface description (table 3-1)	Range			
2. Manning's roughness coefficient, n (table 3-1)	.4			
3. Flow length, L (total L † 300 ft) ... ft	300			
4. Two-year 24-hour rainfall, P_2 ... in	1.31			
5. Land slope, s ... ft/ft	.2			
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t ... hr	.53	+		= .53

Segment ID				
7. Surface description (paved or unpaved)	Unpaved			
8. Flow length, L ... ft	500			
9. Watercourse slope, s ... ft/ft	.28			
10. Average velocity, V (figure 3-1) ... ft/s	8.5			
11. $T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	.02	+		= .02

Segment ID				
12. Cross sectional flow area, a ... ft^2	3			
13. Wetted perimeter, p_w ... ft	6.32			
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ... ft	.474			
15 Channel slope, s ... ft/ft	.2			
16. Manning's roughness coefficient, n	.03			
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ... ft/s	13.5			
18. ~~Flow length~~, L ... ft	10,500			
19. $T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	.22	+		= .22
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) ... Hr				.77

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project: Allied Waste – Wasatch Regional	By: Gordon Jones	Date: 11/4/04
Location: SB9	Checked	Date

Check one: ☒ Present ☐ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

	Segment ID			
1.	Surface description (table 3-1)	Range		
2.	Manning's roughness coefficient, n (table 3-1)	.4		
3.	Flow length, L (total L † 300 ft) ... ft	300		
4.	Two-year 24-hour rainfall, P_2 ... in	1.31		
5.	Land slope, s ... ft/ft	.03		
6.	$T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{\,0.5}\, s^{0.4}}$ Compute T_t ... hr	1.15	+	= 1.15

	Segment ID			
7.	Surface description (paved or unpaved)	unpaved		
8.	Flow length, L ... ft	3800		
9.	Watercourse slope, s ... ft/ft	.11		
10.	Average velocity, V (figure 3-1) ... ft/s	5.2		
11.	$T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	.20	+	= .20

	Segment ID			
12.	Cross sectional flow area, a ... ft^2			
13.	Wetted perimeter, p_w ... ft			
14.	Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ... ft			
15	Channel slope, s ... ft/ft			
16.	Manning's roughness coefficient, n ...			
17.	$V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ... ft/s			
18.	~~Flow~~ length, L ... ft			
19.	$T_t = \frac{L}{3600\ V}$ Compute T_t ... hr		+	= 0
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) ... Hr			1.35

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project Allied Waste - Wasatch Regional	By	Date 11/3/04
Location SB11	Checked	Date

Check one: ☒ Present ☐ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Segment ID				
1. Surface description (table 3-1)	Range			
2. Manning's roughness coefficient, n (table 3-1)	.4			
3. Flow length, L (total L † 300 ft) ft	300			
4. Two-year 24-hour rainfall, P_2 in	1.31			
5. Land slope, s ft/ft	.2			
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t .... hr	.54	+		= .54

Segment ID				
7. Surface description (paved or unpaved)	Unpaved			
8. Flow length, L ft	4700			
9. Watercourse slope, s ft/ft	.125			
10. Average velocity, V (figure 3-1) ft/s	5.5			
11. $T_t = \frac{L}{3600\ V}$ Compute T_t .... hr	.24	+		= .24

Segment ID				
12. Cross sectional flow area, a ft^2				
13. Wetted perimeter, p_w ft				
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ft				
15 Channel slope, s ft/ft				
16. Manning's roughness coefficient, n				
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ft/s				
18. ~~Flow length~~, L ft				
19. $T_t = \frac{L}{3600\ V}$ Compute T_t .... hr		+		= 0
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) Hr				.78

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project Allied Waste - Wasatch Regional	By [illegible]	Date 11/3/[illegible]
Location SB13	Checked	Date

Check one: ☒ Present ☐ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Segment ID				
1. Surface description (table 3-1)	Range			
2. Manning's roughness coefficient, n (table 3-1)	.40			
3. Flow length, L (total L † 300 ft) .... ft	300			
4. Two-year 24-hour rainfall, P_2 .... in	1.31			
5. Land slope, s .... ft/ft	.25			
6. $T_t = \dfrac{0.007\ (nL)^{0.8}}{P_2^{\,0.5}\ s^{0.4}}$ Compute T_t .... hr	.49	+		= .49

Segment ID				
7. Surface description (paved or unpaved)	Unpaved			
8. Flow length, L ....ft	1000			
9. Watercourse slope, s .... ft/ft	.45			
10. Average velocity, V (figure 3-1) .... ft/s	10.5			
11. $T_t = \dfrac{L}{3600\ V}$ Compute T_t .... hr	.03	+		= .03

Segment ID				
12. Cross sectional flow area, a .... ft^2				
13. Wetted perimeter, p_w .... ft				
14. Hydraulic radius, $r = \dfrac{a}{p_w}$ Compute r .... ft				
15 Channel slope, s .... ft/ft				
16. Manning's roughness coefficient, n				
17. $V = \dfrac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ....ft/s				
18. ~~Flow length~~, L .... ft				
19. $T_t = \dfrac{L}{3600\ V}$ Compute T_t .... hr		+		= 0
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) .... Hr				.52

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project	By	Date
Allied Waste - Wasatch Regional	Gordon Jones	11/4/04
Location	**Checked**	**Date**
SB3		

Check one: ☒ Present ☐ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

Segment ID				
1. Surface description (table 3-1)	Range			
2. Manning's roughness coefficient, n (table 3-1)	.41			
3. Flow length, L (total L † 300 ft) .... ft	300			
4. Two-year 24-hour rainfall, P_2 .... in	1.31			
5. Land slope, s .... ft/ft	.08			
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t .... hr	.77	+		= .77

Shallow concentrated flow

Segment ID				
7. Surface description (paved or unpaved)	unpaved			
8. Flow length, L ....ft	700			
9. Watercourse slope, s .... ft/ft	.26			
10. Average velocity, V (figure 3-1) .... ft/s	8.1			
11. $T_t = \frac{L}{3600\ V}$ Compute T_t .... hr	.02	+		= .02

Channel flow

Segment ID				
12. Cross sectional flow area, a .... ft^2				
13. Wetted perimeter, p_w .... ft				
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r .... ft				
15 Channel slope, s .... ft/ft				
16. Manning's roughness coefficient, n				
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ....ft/s				
18. Flow length, L .... ft				
19. $T_t = \frac{L}{3600\ V}$ Compute T_t .... hr		+		= 0
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) .... Hr				.79

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project	By	Date
Allied Waste - Wasatch Regional	Gordon Jones	11/4/04
Location	Checked	Date
SB8		

Check one: ☒ Present ☐ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

	Segment ID			
1.	Surface description (table 3-1)	Range		
2.	Manning's roughness coefficient, n (table 3-1)	.4		
3.	Flow length, L (total L † 300 ft) ... ft	300		
4.	Two-year 24-hour rainfall, P_2 ... in	1.31		
5.	Land slope, s ... ft/ft	.08		
6.	$T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{0.5}\ s^{0.4}}$ Compute T_t ... hr	.77	+	= .77

	Segment ID			
7.	Surface description (paved or unpaved)	unpaved		
8.	Flow length, L ... ft	1700		
9.	Watercourse slope, s ... ft/ft	.28		
10.	Average velocity, V (figure 3-1) ... ft/s	8.5		
11.	$T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	.06	+	= .06

	Segment ID			
12.	Cross sectional flow area, a ... ft^2			
13.	Wetted perimeter, p_w ... ft			
14.	Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ... ft			
15	Channel slope, s ... ft/ft			
16.	Manning's roughness coefficient, n ...			
17.	$V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ... ft/s			
18.	~~Flow~~ length, L ... ft			
19.	$T_t = \frac{L}{3600\ V}$ Compute T_t ... hr		+	= 0
20.	Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) ... Hr			.83

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project	By	Date
Allied Waste – Wasatch Regional	Gordon Jones	11/4/04
Location	**Checked**	**Date**
SB2		

Check one: ☒ Present ☐ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

Segment ID				
1. Surface description (table 3-1)	Range			
2. Manning's roughness coefficient, n (table 3-1)	.4[a]			
3. Flow length, L (total L † 300 ft) ... ft	300			
4. Two-year 24-hour rainfall, P_2 ... in	1.31			
5. Land slope, s ... ft/ft	.05			
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t ... hr	.93	+		= .93

Segment ID				
7. Surface description (paved or unpaved)	Unpaved			
8. Flow length, L ... ft	4600			
9. Watercourse slope, s ... ft/ft	.12			
10. Average velocity, V (figure 3-1) ... ft/s	5.5			
11. $T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	.23	+		= .23

Segment ID				
12. Cross sectional flow area, a ... ft^2				
13. Wetted perimeter, p_w ... ft				
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ... ft				
15 Channel slope, s ... ft/ft				
16. Manning's roughness coefficient, n ...				
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ... ft/s				
18. ~~Flow~~ length, L ... ft				
19. $T_t = \frac{L}{3600\ V}$ Compute T_t ... hr		+		= 0
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) ... Hr				1.16

Salt Lake City Corporation
Department of Public Utilities

27-Jul-04

Frank Hamilton, P.E.

1530 South West Temple, Salt Lake City, UT 84115

(801) 483-6790 | Fax (801) 483-6847

FAX TRANSMITTAL

TO: Greg Poole
HAL

FAX: (801) 566-5581

SUBJECT: **SLC Hydrology Manual**
ARF

Comments:

Thanks,

Frank

Number of Pages including this one: 4

2.5 *Long-Duration Elevation Adjustments*

Long duration (6-hours and over) DDF statistics show a general increase of rainfall intensity with elevation to the top of the Wasatch Range. A linear adjustment of rainfall was consistent with gage DDF data and NOAA Atlas 2 (ref. 20, 41). In the preprocessor, these adjustments were determined for elevations below 6,000 ft and may underestimate long-duration precipitation for elevations over 6,000 ft. Adjustment factors are shown below:

5-Year: +15% per 1000 ft above 4226
10-Year: +13% per 1000 ft above 4226
25-Year: +12% per 1000 ft above 4226
100-Year: +11% per 1000 ft above 4226

2.6 *Areal Reduction Factors (ARF)*

Point precipitation gage statistics are only representative of areas of a few hundred acres. The distance for significant correlation between point gage measurements is characteristically a few miles for short-duration precipitation (less than one hour) and up to a few hundred miles for long-duration precipitation. Relationships for correcting point gage intensity to mean areal intensity have been developed from analyses of storm precipitation from closely spaced gage networks in Illinois, Northeast U.S., Arizona, New Mexico and Southern California (ref. 5, 12, 13, 14, 15, 21, 22, 23, 27, 30, 40, 42).

The U.S. Army Corps of Engineers made a special study of cloudbursts in the Salt Lake County area in 1970-1975 (ref. 30, 31). Their ARF's are similar to those determined for cloudbursts in Southern California and Arizona. The factors developed from the USCOE analysis of Salt Lake cloudbursts are used in the preprocessor for durations up to three hours. For durations over three hours, the NOAA Atlas values are used.

The maximum peak discharge at any given concentration point will normally be computed by entering the total drainage area in the preprocessor. For most studies, a single downstream concentration point will give adequate peak discharge definition for all the concentration points upstream in the model. As the size of the drainage area increases (for values over approximately 100 acres) other concentration points may be necessary. The HEC-1 model should be run for a few selected areas, and peak flows interpolated by drainage area for intermediate points. ARF equations are listed below and illustrated in Table 2:

5-min:	.01*(100-18.5*Area^.46)
10-min:	.01*(100-14.2*Area^.46)
15-min:	.01*(100-12.0*Area^.46)
30-min:	.01*(100-9.2*Area^.46)
1-hr:	.01*(100-7.0*Area^.46)
2-hr:	.01*(100-5.3*Area^.46)

3-hr:	.01*(100-4.5*Area^.46)
6-hr:	.01*(100-3.5*Area^.46)
12-hr:	.01*(100-2.6*Area^.46)
1-day:	.01*(100-2.0*Area^.46)
2-day:	.01*(100-1.5*Area^.46)
3-day:	.01*(100-1.3*Area^.46)

Units of area in the equations above are square miles.

Table 2
Areal Reduction Factors

Duration (min)	Area (Square Miles)											
	.5	1	2	3	4	5	6	7	8	9	10	20
5	.87	.82	.75	.69	.65	.61	.58	.55	.52	.49	.47	.27
10	.90	.86	.80	.76	.73	.70	.68	.65	.63	.61	.59	.44
15	.91	.88	.83	.80	.77	.75	.73	.71	.69	.67	.65	.52
30	.93	.91	.87	.85	.83	.81	.79	.77	.76	.75	.73	.64
60	.95	.93	.90	.88	.87	.85	.84	.83	.82	.81	.80	.72
120	.96	.95	.93	.91	.90	.89	.88	.87	.86	.85	.85	.79
180	.97	.96	.94	.93	.91	.91	.90	.89	.88	.88	.87	.82
360	.97	.97	.95	.94	.93	.93	.92	.91	.91	.90	.90	.86
720	.98	.97	.96	.96	.95	.95	.94	.94	.93	.93	.93	.90
1440	.99	.98	.97	.97	.96	.96	.95	.95	.95	.95	.94	.92
2880	.99	.99	.98	.98	.97	.97	.97	.96	.96	.96	.96	.94
4320	.99	.99	.98	.98	.98	.97	.97	.97	.97	.96	.96	.95

2.7 *Precipitation Temporal Distribution*

,92 -

Time distribution of rainfall within storms is important in estimating flood hydrographs. Distributions vary with storm type (orthographic, convective), intensity and duration. There is no typical distribution that is applicable to all situations. The Farmer-Fletcher (ref. 9) distribution was used for the central hour of the three hour, five-minute time step cloudburst. The remaining two hour intensities were distributed symmetrically around the central hour.

The long-duration three-day, hourly time step storm was provided a balanced symmetrical distribution as shown in ref. 13. A symmetrical precipitation distribution is constructed such that the depths specified for the greatest intensities occur during the central part of the storm. The design storm pattern consists of incremental precipitation depths nested within the storm duration in an alternating pattern with the maximum value in the center and the second highest value to the right of center.

2.8 Constructing A Design Storm

Design storms are created by SLPRE. The preprocessor will adjust the precipitation for the mean elevation of each subbasin. It will also adjust the precipitation for the entire drainage basin area. The preprocessor can generate design storms for 5-,10-, 25-, and 100-year recurrence intervals for durations of three hours or 72 hours.

3. RUNOFF ANALYSIS CRITERIA

3.1 Introduction

The Corps of Engineers computer program, HEC-1, is used for the calculation of flow hydrographs. Use of HEC-1 with a consistent unit graph for all hydrologic calculations will provide compatible results between the calculated peak flows from smaller developments and the larger watershed master plan in which they are located.

In order to minimize the potential for entering inappropriate or inconsistent input data and to aid the user in developing an error-free HEC-1 input file, the HEC-1 Preprocessor, SLPRE is provided. The Preprocessor (SLPRE) uses HEC-1 compatible methods to generate, combine and route hydrographs. The SLPRE program can be used for the analysis of existing drainage areas as well as design of drainage systems.

3.2 HEC-1, Flood Hydrograph Package

HEC-1 was developed by the Corps of Engineers, Hydraulic Engineering Center, Davis, California. HEC-1 is a mathematical watershed model designed to simulate the surface water runoff response of a drainage basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. The result of the modeling process is a computation of stream flow hydrographs at desired locations within the basin.

3.3 Interception/Infiltration

Land surface infiltration, depression storage and interception are referred to in the HEC-1 model as precipitation losses. The initial and constant loss option in HEC-1 is used to calculate losses from pervious areas due to infiltration.

The HEC-1 program uses a LU record to calculate losses. This record includes the initial loss, constant loss rate, and percent of subbasin which is impervious. SLPRE aids in the

STORM WATER CONVEYANCE AND [illegible] DESIGN

HANSEN ALLEN & LUCE inc ENGINEERS	CLIENT:	Wasatch Regional	SHEET 1	OF 2
	PROJECT:	Landfill Permit	COMPUTED:	GLJ
	FEATURE:	Riprap Design	CHECKED:	KCS
	PROJECT NO.:	113.30.100	DATE: December 2004	

Purpose: To determine the rip rap D_{50} size as well as the channel depth requirement for each channel segment.

Method: The two main channels have been divided into segments, 1-A through 1-G and 2-A through 2-E. Hansen, Allen, and Luce (HAL) has developed a spreadsheet called "Trapezoidal Channel Flow Calculations" to calculate the rip rap safety factor and minimum channel depth based on design flow, slope, channel dimensions and an assumed riprap D_{50}.

References: The following materials were used to develop the HAL "Trapezoidal Channel Flow Calculations."

- Abt, S.R. et. al., 1987. "Development of Riprap Design Criteria by Riprap Testing in Flumes: Phase I." Nureg/CR4651, ORNL/TM-10100/V2, U.S. Nuclear Regulatory Commission, Washington, D.C.

- Abt, S.R. et. al., 1988. "Development of Riprap Design Criteria by Riprap Testing in Flumes: Phase II-Follow-Up Investigation." Nureg/CR4651, ORNL/TM-10100/V2, U.S. Nuclear Regulatory Commission, Washington, D.C.

- Anderson, A.G., A.S. Paintal, and J.T. Davenport, 1970. "Tentative Design Procedure for Riprap Lined Channels." NCHRP Rep. 108, Hwy. Res. Board, National Academy Of Science-National Academy of Engineering., Washington D.C.

- Haan, C.T., B.J. Barfield and J.C. Hayes, "Design Hydrology and Sedimentology for Small Catchments", Academic Press.

- Jarrett, R.D., 1984. "Hydraulics of High Gradient Streams." ASCE Journal of Hydraulic Engineering.

- Rice, C.E. et. al., February 1998. "Roughness of Loose Rock Riprap on Steep Slopes". ASCE Journal of Hydraulic Engineering.

- US Department of Transportation, April 1988. "Design of Roadside Channels with Flexible Linings". Washington, D.C.

- Wang, Sany-yi and Hsieh Wen Shen, March 1985. "Incipient Sediment Motion and Riprap Design" ASCE Journal of Hydraulic Engineering, Vol. III, No. 3 Paper No. 19562.

Results: The results are summarized in the table below and the spreadsheet can be found in the calculation sheets.

Riprap Design

Channel Segment	Slope	Peak Design Flow (CFS)	Rip Rap D_{50} Size (ft)	Min Depth (ft)
Channel 1-A	0.25%	303	0.33	4.30
Channel 1-B	1.00%	303	1.0	4.03
Channel 1-C	5.00%	368	2.5	4.02
Channel 1-D	2.00%	368	1.75	4.19
Channel 1-E	0.25%	379	0.33	4.67
Channel 1-F	5.00%	551	2.75	4.80
Channel 1-G	1.00%	551	1.17	5.20
Channel 2-A	0.25%	63	0.25	2.51
Channel 2-B	2.00%	86	1.0	2.54
Channel 2-C	5.00%	86	1.75	2.46
Channel 2-D	15.00%	86	2.5	2.36
Channel 2-E	1.5%	86	0.75	2.60

HANSEN ALLEN & LUCE inc	Client: *Wasatch Regional*	Sheet	*1 of 2*
	Project: *Landfill Permit*	Comp.	*GLJ*
	Feature: *Run-on Channel 1-A*	Chck'd	*KCS*
	Project #: *113.30.100*	Date	*17-Dec-04*

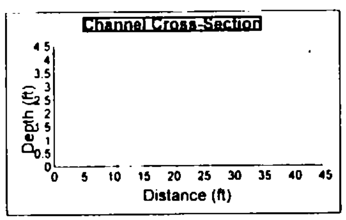
Trapezoidal Channel Flow Calculations

GENERAL CRITERIA:

	Design Flow:	303.00	cfs
	Bottom Width:	15.0	feet
	Side Slope1:	2.5	m1
	Side Slope2:	4.0	m2
	Friction Factor:		
	Assumed D50:	0.33	
Anderson et al. (1970)	If X = 1, n = $0.0395(D50)^{1/6}$		
Abt et al (1987, 1988)	If X = 2, n = $0.0456(D50*S)^{0.159}$		
	If X = 3, n = $\{D50^{1/6}*(R/D50)^{1/6}\}/\{3.82*[2.25+5.23*LOG(R/D50)]\}$		
	Generally Applicable for R/D50 > 0.5		
Jarrett (1984)	If X = 4, n = $0.39*(S^{0.38})*(R^{-0.16})$		
	If X = 5, n = input n value		
	X:	4.0	
	Input n Value when X = 5:		
	Min. Bottom Slope	0.002	ft/ft
	Max. Bottom Slope:	0.003	ft/ft
	Freeboard:	1.0	feet
Depth Check	Depth (Min. Slope):	3.2	feet
	$Q\text{-}1.49AR^{(2/3)}S^{(1/2)}/n=$	[illegible]	Accuracy
	Calc (used) n Value:	[illegible]	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity:	[illegible]	ft/sec
	Froude Number:	[illegible]	
Velocity Check:	Depth (Max. Slope):	3.3	feet
	$Q\text{-}1.49AR^{(2/3)}S^{(1/2)}/n=$	[illegible]	Accuracy
	Calc (used) n Value:	[illegible]	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity:	[illegible]	ft/sec
	Froude Number:	[illegible]	

Channel Design Summary:

Bottom Width:	[illegible]	feet
Side Slope1	[illegible]	1/m1
Side Slope2:		1/m2
Min. Bottom Slope	[illegible]	ft/ft
Max. Bottom Slope:		ft/ft
Min. Channel Depth:		feet
Channel Top Width:		feet



HANSEN ALLEN & LUCE inc	Client: *Wasatch Regional*	Sheet	*2 of 2*
	Project: *Landfill Permit*	Comp.	*GLJ*
	Feature: *Run-on Channel 1-A*	Chck'd	*KCS*
	Project #: *113.30.100*	Date	*17-Dec-04*

DESIGN CRITERIA:

Design Flow:	[illegible]		cfs
Bottom Width:	[illegible]		feet
Side Slope1:	[illegible]		1/m1
Side Slope2:	[illegible]		1/m2
Friction Factor (Min. S & Max S):	[illegible]	[illegible]	
Min. Bottom Slope:	[illegible] %		
Max. Bottom Slope:	[illegible] %		
Flow Depth (Min. S):	[illegible]		feet
Flow Depth (Max. S):	[illegible]		feet
Angle Repose (Ar):	39.0		degrees
Specific Gravity	2.55		

Reynolds No. = U*D50/v, where U = Shear Velocity, v = viscosity

U = (gRS)^0.5 for Smin

Reynolds # for Smin [illegible]

U = $(gRS)^{0.5}$ for Smax

Reynolds # for Smax

T = G*d*S where G = Unit weight of Water

Nb = F*T/(G(SD-1)D50)

F = (1/0.047) = 21.3 for flat slopes with Reynolds No. < 500

F = (1/0.062) = 16.1 for 500 < Reynolds No. < 40,000

F = varies from (1/0.062) = 16.1 for Reynolds No. = 40,000 to (1/0.25) = 4 for Reynolds No. = 500,000 or larger

K for S min (Compare K vs. R Chart)	[illegible]
K for S max (Compare K vs. R Chart)	[illegible]
F for S min	[illegible]
F for S max	[illegible]

SFb = (Cos a tan b)/(sin a + Nb tan b)

Tmax = K*G*d*S

Set K = 0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope

K: 0.80

Ns = F*Tmax/(G(SG-1)D)

A = Atan(1/m)

B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar)) + Sin(Ar))

Nsp = Ns(1 + Sin(Ar + B)/2)

SFs = Cos(A)Tan(Ar)/(nTan(Ar) + Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	0.33	0.33	feet
T	[illegible]	[illegible]	lb/ft2
Nb	[illegible]	[illegible]	
Tmax	[illegible]	[illegible]	lb/ft2
Ns	[illegible]	[illegible]	
m Critical	[illegible]	[illegible]	
A (m crit)	[illegible]	[illegible]	degrees
B	[illegible]	[illegible]	degrees
Nsp	[illegible]	[illegible]	
SFb	[illegible]		
SFs	[illegible]	[illegible]	

HANSEN ALLEN & LUCE inc

Client	*Wasatch Regional*	Sheet	*1 of 2*
Project	*Landfill Permit*	Comp.	*GLJ*
Feature	*Run-on Channel 1-B*	Chck'd	*KCS*
Project #	*113.30.100*	Date	*17-Dec-04*

Trapezoidal Channel Flow Calculations

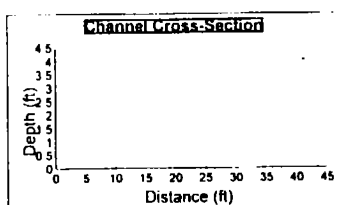
GENERAL CRITERIA:

	Design Flow:	303.00	cfs
	Bottom Width:	15.0	feet
	Side Slope1:	2.5	m1
	Side Slope2	4.0	m2
	Friction Factor:		
	Assumed D50:	1.00	
Anderson et al (1970)	If X = 1, $n = 0.0395(D50)^{1/6}$		
Abt et al (1987, 1988)	If X = 2, $n = 0.0456(D50*S)^{0.159}$		
	If X = 3, $n = \{D50^{1/6}*(R/D50)^{1/6}\}/\{3.82*[2.25+5.23*LOG(R/D50)]\}$		
	Generally Applicable for R/D50 > 0.5		
Jarrett (1984)	If X = 4, $n = 0.39*(S^{0.38})*(R^{-0.16})$		
	If X = 5, n = input n value		
	X:	4.0	
	Input n Value when X = 5.		
	Min. Bottom Slope:	0.009	ft/ft
	Max. Bottom Slope:	0.011	ft/ft
	Freeboard:	1.0	feet

Depth Check	Depth (Min. Slope):	3.0	feet
	$Q - 1.49AR^{(2/3)}S^{(1/2)}/n =$	[illegible]	Accuracy
	Calc (used) n Value:	[illegible]	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity:	[illegible]	ft/sec
	Froude Number:	[illegible]	
Velocity Check	Depth (Max. Slope):	3.0	feet
	$Q - 1.49AR^{(2/3)}S^{(1/2)}/n =$	[illegible]	Accuracy
	Calc (used) n Value:	[illegible]	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity:	[illegible]	ft/sec
	Froude Number:	[illegible]	

Channel Design Summary:

Bottom Width:	feet
Side Slope1.	1/m1
Side Slope2	1/m2
Min. Bottom Slope:	ft/ft
Max. Bottom Slope.	ft/ft
Min. Channel Depth	feet
Channel Top Width	feet



HANSEN ALLEN & LUCE inc

Client	*Wasatch Regional*	Sheet	*2 of 2*
Project	*Landfill Permit*	Comp.	*GLJ*
Feature	*Run-on Channel 1-B*	Chck'd	*KCS*
Project #	*113.30.100*	Date	*17-Dec-04*

DESIGN CRITERIA:

Design Flow:	303.00	cfs
Bottom Width:	15.00	feet
Side Slope1:	2.50	1/m1
Side Slope2:	[illegible]	1/m2
Friction Factor (Min. S & Max. S):	[illegible]	[illegible]
Min. Bottom Slope:	[illegible]	%
Max. Bottom Slope:	[illegible]	%
Flow Depth (Min. S):	[illegible]	feet
Flow Depth (Max. S):	[illegible]	feet
Angle Repose (Ar):	39.0	degrees
Specific Gravity	2.55	

Reynolds No. = U*D50/v, where U = Shear Velocity, v = viscosity

U = (gRS)^0.5 for Smin

Reynolds # for Smin

$U = (gRS)^{0.5}$ for Smax

Reynolds # for Smax

T = G*d*S where G = Unit weight of Water

Nb = F*T/(G(SD-1)D50)

F = (1/0.047) = 21.3 for flat slopes with Reynolds No. < 500

F = (1/0.062) = 16.1 for 500 < Reynolds No. < 40,000

F = varies from (1/0.062) = 16.1 for Reynolds No. = 40,000 to (1/0.25) = 4 for Reynolds No. = 500,000 or larger

K for S min (Compare K vs. R Chart)

K for S max (Compare K vs. R Chart)

F for S min

F for S max

SFb = (Cos a tan b)/(sin a + Nb tan b)

Tmax = K*G*d*S

Set K = 0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope

K: 0.80

Ns = F*Tmax/(G(SG-1)D)

A = Atan(1/m)

B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar)) + Sin(Ar))

Nsp = Ns(1 + Sin(Ar + B)/2)

SFs = Cos(A)Tan(Ar)/(nTan(Ar) + Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	1.00	1.00	feet
T	[illegible]	[illegible]	lb/ft2
Nb	[illegible]	[illegible]	
Tmax	[illegible]	[illegible]	lb/ft2
Ns	[illegible]	[illegible]	
m Critical	[illegible]	[illegible]	
A (m crit)	[illegible]	[illegible]	degrees
B	[illegible]	[illegible]	degrees
Nsp			
SFb	[illegible]		
SFs	[illegible]		

HANSEN ALLEN & LUCE inc	Client: Wasatch Regional	Sheet: 1 of 2
	Project: Landfill Permit	Comp.: GLJ
	Feature: Run-on Channel 1-C	Chck'd: KCS
	Project #: 113.30.100	Date: 17-Dec-04

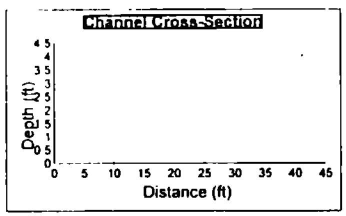
Trapezoidal Channel Flow Calculations

GENERAL CRITERIA:

	Design Flow:	368.00	cfs
	Bottom Width:	15.0	feet
	Side Slope1:	2.5	m1
	Side Slope2:	4.0	m2
	Friction Factor:		
	Assumed D50:	2.50	
Anderson et al (1970)	If X = 1, $n = 0.0395(D50)^{1/6}$		
Abt et al (1987, 1988)	If X = 2, $n = 0.0456(D50*S)^{0.159}$		
	If X = 3, $n = \{D50^{1/6}*(R/D50)^{1/6}\}/\{3.82*[2.25+5.23*LOG(R/D50)]\}$		
	Generally Applicable for R/D50 > 0.5		
Jarrett (1984)	If X = 4, $n = 0.39*(S^{0.38})*(R^{-0.16})$		
	If X = 5, n = input n value		
	X:	4.0	
	Input n Value when X = 5:		
	Min. Bottom Slope:	0.019	ft/ft
	Max. Bottom Slope:	0.051	ft/ft
	Freeboard:	1.0	feet
Depth Check	Depth (Min. Slope):	3.0	feet
	$Q - 1.49AR^{(2/3)}S^{(1/2)}/n =$		Accuracy
	Calc (used) n Value:		
	Required Depth:		feet
	Area:		ft2
	Perimeter:		feet
	Hydraulic Radius:		feet
	Velocity:		ft/sec
	Froude Number:		
Velocity Check	Depth (Max. Slope):	3.0	feet
	$Q - 1.49AR^{(2/3)}S^{(1/2)}/n =$		Accuracy
	Calc (used) n Value:		
	Required Depth:		feet
	Area:		ft2
	Perimeter:		feet
	Hydraulic Radius:		feet
	Velocity:		ft/sec
	Froude Number:		

Channel Design Summary:

Bottom Width		feet
Side Slope1		1/m1
Side Slope2		1/m2
Min. Bottom Slope		ft/ft
Max. Bottom Slope:		ft/ft
Min. Channel Depth:		feet
Channel Top Width:		feet



HANSEN ALLEN & LUCE inc	Client: Wasatch Regional	Sheet: 2 of 2
	Project: Landfill Permit	Comp.: GLJ
	Feature: Run-on Channel 1-C	Chck'd: KCS
	Project #: 113.30.100	Date: 17-Dec-04

DESIGN CRITERIA:

Design Flow:		cfs
Bottom Width:		feet
Side Slope1:		1/m1
Side Slope2:		1/m2
Friction Factor (Min. S & Max. S):		
Min. Bottom Slope:		%
Max. Bottom Slope:		%
Flow Depth (Min. S):		feet
Flow Depth (Max. S):		feet
Angle Repose (Ar):	39.0	degrees
Specific Gravity	2.55	

Reynolds No. = U*D50/v, where U = Shear Velocity, v = viscosity
U = (gRS)^0.5 for Smin
Reynolds # for Smin
$U = (gRS)^{0.5}$ for Smax
Reynolds # for Smax
T = G*d*S where G = Unit weight of Water
Nb = F*T/(G(SD-1)D50)
F = (1/0.047) = 21.3 for flat slopes with Reynolds No. < 500
F = (1/0.062) = 16.1 for 500 < Reynolds No. < 40,000
F = varies from (1/0.062) = 16.1 for Reynolds No. = 40,000 to (1/0.25) = 4 for Reynolds No. = 500,000 or larger
K for S min (Compare K vs. R Chart)
K for S max (Compare K vs. R Chart)
F for S min
F for S max
SFb = (Cos a tan b)/(sin a + Nb tan b)
Tmax = K*G*d*S
Set K = 0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope
K: 0.80
Ns = F*Tmax/(G(SG-1)D)
A = Atan(1/m)
B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar)) + Sin(Ar))
Nsp = Ns(1 + Sin(Ar + B)/2)
SFs = Cos(A)Tan(Ar)/(nTan(Ar) + Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	2.50	2.50	feet
T			lb/ft2
Nb			
Tmax			lb/ft2
Ns			
m Critical			
A (m crit)			degrees
B			degrees
Nsp			
SFb			
SFs			

HANSEN ALLEN & LUCE inc

Client	*Wasatch Regional*	Sheet	*1 of 2*
Project	*Landfill Permit*	Comp.	*GLJ*
Feature	*Run-on Channel 1-D*	Chck'd	*KCS*
Project #	*113.30.100*	Date	*17-Dec-04*

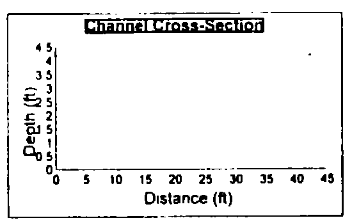
Trapezoidal Channel Flow Calculations

GENERAL CRITERIA:

	Design Flow:	368.00	cfs
	Bottom Width:	15.0	feet
	Side Slope1:	2.5	m1
	Side Slope2:	4.0	m2
	Friction Factor:		
	Assumed D50:	1.75	
Anderson et al. (1970)	If X = 1, n = 0.0395(D50)$^{1/6}$		
Abt et al (1987, 1988)	If X = 2, n = 0.0456(D50*S)$^{0.159}$		
	If X = 3, n = {D50$^{1/6}$*(R/D50)$^{1/6}$}/{3.82*[2.25+5.23*LOG(R/D50)]}		
	Generally Applicable for R/D50 > 0.5		
Jarrett (1984)	If X = 4, n = 0.39*(S$^{0.38}$)*(R$^{0.16}$)		
	If X = 5, n = input n value		
	X:	1.0	
	Input n Value when X = 5.		
	Min. Bottom Slope:	0.019	ft/ft
	Max. Bottom Slope:	0.021	ft/ft
	Freeboard:	1.0	feet
Depth Check	Depth (Min. Slope):	3.2	feet
	Q-1.49AR$^{(2/3)}$S$^{(1/2)}$/n =	[illegible]	Accuracy
	Calc (used) n Value:	[illegible]	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity:	[illegible]	ft/sec
	Froude Number:	[illegible]	
Velocity Check	Depth (Max. Slope):	3.2	feet
	Q-1.49AR$^{(2/3)}$S$^{(1/2)}$/n =	[illegible]	Accuracy
	Calc (used) n Value:	[illegible]	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity:	[illegible]	ft/sec
	Froude Number:	[illegible]	

Channel Design Summary:

Bottom Width:	feet
Side Slope1:	1/m1
Side Slope2:	1/m2
Min. Bottom Slope:	ft/ft
Max. Bottom Slope:	ft/ft
Min. Channel Depth:	feet
Channel Top Width:	feet



HANSEN ALLEN & LUCE inc

Client	*Wasatch Regional*	Sheet	*2 of 2*
Project	*Landfill Permit*	Comp	*GLJ*
Feature	*Run-on Channel 1-D*	Chck'd	*KCS*
Project #	*113.30.100*	Date	*17-Dec-04*

DESIGN CRITERIA:

Design Flow:	[illegible]		cfs
Bottom Width:	[illegible]		feet
Side Slope1:	[illegible]		1/m1
Side Slope2:	[illegible]		1/m2
Friction Factor (Min. S & Max. S):	[illegible]	[illegible]	
Min. Bottom Slope:	[illegible]	%	
Max. Bottom Slope:	[illegible]	%	
Flow Depth (Min. S):	[illegible]		feet
Flow Depth (Max. S):	[illegible]		feet
Angle Repose (Ar):	39.0		degrees
Specific Gravity	2.55		

Reynolds No. = U*D50/v, where U = Shear Velocity, v = viscosity

U = (gRS)^0.5 for Smin [illegible]

Reynolds # for Smin [illegible]

U = (gRS)$^{0.5}$ for Smax

Reynolds # for Smax

T = G*d*S where G = Unit weight of Water

Nb = F*T/(G(SD-1)D50)

F = (1/0.047) = 21.3 for flat slopes with Reynolds No. < 500

F = (1/0.062) = 16.1 for 500 < Reynolds No. < 40,000

F = varies from (1/0.062) = 16.1 for Reynolds No. = 40,000 to (1/0.25) = 4 for Reynolds No. = 500,000 or larger

K for S min (Compare K vs. R Chart) [illegible]

K for S max (Compare K vs. R Chart)

F for S min

F for S max

SFb = (Cos a tan b)/(sin a + Nb tan b)

Tmax = K*G*d*S

Set K = 0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope

K: 0.80

Ns = F*Tmax/(G(SG-1)D)

A = Atan(1/m)

B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar)) + Sin(Ar))

Nsp = Ns(1 + Sin(Ar + B)/2)

SFs = Cos(A)Tan(Ar)/(nTan(Ar) + Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	1.75	1.75	feet
T	[illegible]	[illegible]	lb/ft2
Nb	[illegible]	[illegible]	
Tmax	[illegible]	[illegible]	lb/ft2
Ns	[illegible]	[illegible]	
m Critical	[illegible]	[illegible]	
A (m crit)	[illegible]	[illegible]	degrees
B	[illegible]	[illegible]	degrees
Nsp	[illegible]	[illegible]	
SFb	[illegible]	[illegible]	
SFs	[illegible]	[illegible]	

HANSEN ALLEN & LUCE inc ENGINEERS

Client	Wasatch Regional	Sheet	1 of 2
Project	Landfill Permit	Comp.	GLJ
Feature	Run-on Channel 1-E	Chck'd	KCS
Project #	113.30.100	Date	17-Dec-04

Trapezoidal Channel Flow Calculations

GENERAL CRITERIA:

Design Flow:	379.00	cfs
Bottom Width:	15.0	feet
Side Slope1:	2.5	m1
Side Slope2:	4.0	m2
Friction Factor:		
Assumed D50:	0.33	

Anderson et al (1970) If X = 1, n = 0.0395(D50)$^{1/6}$

Abt et al (1987, 1988) If X = 2, n = 0.0456(D50*S)$^{0.159}$

If X = 3, n = {D50$^{1/6}$*(R/D50)$^{1/6}$}/{3.82*[2.25+5.23*LOG(R/D50)]}

Generally Applicable for R/D50 > 0.5

Jarrett (1984) If X = 4, n = 0.39*(S$^{0.38}$)*(R$^{-0.16}$)

If X = 5, n = input n value

X:	4.0	
Input n Value when X = 5:		
Min. Bottom Slope:	0.002	ft/ft
Max. Bottom Slope:	0.003	ft/ft
Freeboard:	1.0	feet

Depth Check

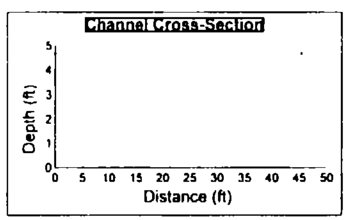
Depth (Min. Slope):	3.7	feet
Q-1.49AR$^{(2/3)}$S$^{(1/2)}$/n =	[illegible]	Accuracy
Calc (used) n Value:	[illegible]	
Required Depth:	[illegible]	feet
Area:	[illegible]	ft2
Perimeter:	[illegible]	feet
Hydraulic Radius:	[illegible]	feet
Velocity:	[illegible]	ft/sec
Froude Number:	[illegible]	

Velocity Check

Depth (Max. Slope):	3.6	feet
Q-1.49AR$^{(2/3)}$S$^{(1/2)}$/n =	[illegible]	Accuracy
Calc (used) n Value:	[illegible]	
Required Depth:	[illegible]	feet
Area:	[illegible]	ft2
Perimeter:	[illegible]	feet
Hydraulic Radius:	[illegible]	feet
Velocity:	[illegible]	ft/sec
Froude Number:	[illegible]	

Channel Design Summary:

Bottom Width:	feet
Side Slope1:	1/m1
Side Slope2:	1/m2
Min. Bottom Slope:	ft/ft
Max. Bottom Slope:	ft/ft
Min. Channel Depth:	feet
Channel Top Width:	feet



HANSEN ALLEN & LUCE inc ENGINEERS

Client	Wasatch Regional	Sheet	2 of 2
Project	Landfill Permit	Comp.	GLJ
Feature	Run-on Channel 1-E	Chck'd	KCS
Project #	113.30.100	Date	17-Dec-04

DESIGN CRITERIA:

Design Flow:	[illegible]	cfs
Bottom Width:	[illegible]	feet
Side Slope1:	[illegible]	1/m1
Side Slope2:	[illegible]	1/m2
Friction Factor (Min. S & Max. S):	[illegible]	[illegible]
Min. Bottom Slope:	[illegible]	%
Max. Bottom Slope:	[illegible]	%
Flow Depth (Min. S):	[illegible]	feet
Flow Depth (Max. S):	[illegible]	feet
Angle Repose (Ar):	39.0	degrees
Specific Gravity	2.55	

Reynolds No. = U*D50/v, where U = Shear Velocity, v = viscosity

U = (gRS)^0.5 for Smin

Reynolds # for Smin [illegible]

U = (gRS)$^{0.5}$ for Smax [illegible]

Reynolds # for Smax [illegible]

T = G*d*S where G = Unit weight of Water

Nb = F*T/(G(SG-1)D50)

F = (1/0.047) = 21.3 for flat slopes with Reynolds No. < 500

F = (1/0.062) = 16.1 for 500 < Reynolds No. < 40,000

F = varies from (1/0.062) = 16.1 for Reynolds No. = 40,000 to (1/0.25) = 4 for Reynolds No. = 500,000 or larger

K for S min (Compare K vs. R Chart) [illegible]

K for S max (Compare K vs. R Chart) [illegible]

F for S min [illegible]

F for S max [illegible]

SFb = (Cos a tan b)/(sin a + Nb tan b)

Tmax = K*G*d*S

Set K = 0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope

K: 0.80

Ns = F*Tmax/(G(SG-1)D)

A = Atan(1/m)

B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar))+Sin(Ar))

Nsp = Ns(1+Sin(Ar+B)/2)

SFs = Cos(A)Tan(Ar)/(nTan(Ar)+Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	0.33	0.33	feet
T	[illegible]	[illegible]	lb/ft2
Nb	[illegible]	[illegible]	
Tmax	[illegible]	[illegible]	lb/ft2
Ns	[illegible]	[illegible]	
m Critical	[illegible]	[illegible]	
A (m crit)	[illegible]	[illegible]	degrees
B	[illegible]	[illegible]	degrees
Nsp		[illegible]	
SFb			
SFs	[illegible]		

HANSEN ALLEN & LUCE inc	Client: *Wasatch Regional*	Sheet	*1 of 2*
	Project: *Landfill Permit*	Comp	*GLJ*
	Feature: *Run-on Channel 1-F*	Chck'd	*KCS*
	Project #: *113.30.100*	Date	*17-Dec-04*

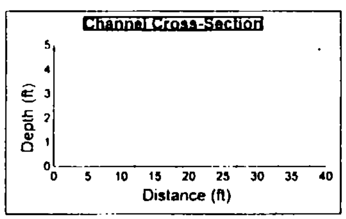
Trapezoidal Channel Flow Calculations

GENERAL CRITERIA:

	Design Flow:	551.00	cfs
	Bottom Width:	15.0	feet
	Side Slope1:	2.5	m1
	Side Slope2:	2.5	m2
	Friction Factor:		
	Assumed D50:	2.75	
Anderson et al. (1970)	If X=1, $n=0.0395(D50)^{1/6}$		
Abt et al. (1987, 1988)	If X=2, $n=0.0456(D50*S)^{0.159}$		
	If X=3, $n=\{D50^{1/6}*(R/D50)^{1/6}\}/\{3.82*[2.25+5.23*LOG(R/D50)]\}$		
	Generally Applicable for R/D50 > 0.5		
Jarrett (1984)	If X=4, $n=0.39*(S^{0.38})*(R^{-0.16})$		
	If X=5, n=input n value		
	X:	4.0	
	Input n Value when X=5:		
	Min. Bottom Slope:	0.049	ft/ft
	Max. Bottom Slope:	0.051	ft/ft
	Freeboard:	1.0	feet
Depth Check	Depth (Min. Slope):	3.8	feet
	$Q-1.49AR^{(2/3)}S^{(1/2)}/n=$	[illegible]	Accuracy
	Calc (used) n Value:		
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity:	[illegible]	ft/sec
	Froude Number:	[illegible]	
Velocity Check	Depth (Max. Slope):	3.8	feet
	$Q-1.49AR^{(2/3)}S^{(1/2)}/n=$	[illegible]	Accuracy
	Calc (used) n Value:	0.108	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity	[illegible]	ft/sec
	Froude Number:	[illegible]	

Channel Design Summary:

Bottom Width:		feet
Side Slope1		1/m1
Side Slope2		1/m2
Min. Bottom Slope:		ft/ft
Max. Bottom Slope:		ft/ft
Min. Channel Depth:		feet
Channel Top Width:		feet



HANSEN ALLEN & LUCE inc	Client: *Wasatch Regional*	Sheet	*2 of 2*
	Project: *Landfill Permit*	Comp	*GLJ*
	Feature: *Run-on Channel 1-F*	Chck'd	*KCS*
	Project #: *113.30.100*	Date	*17-Dec-04*

DESIGN CRITERIA:

Design Flow:	[illegible]	cfs
Bottom Width:	[illegible]	feet
Side Slope1:	[illegible]	1/m1
Side Slope2:	[illegible]	1/m2
Friction Factor (Min. S & Max. S):	[illegible]	[illegible]
Min. Bottom Slope:	[illegible]	%
Max. Bottom Slope:	[illegible]	%
Flow Depth (Min. S):	[illegible]	feet
Flow Depth (Max. S):	[illegible]	feet
Angle Repose (Ar):	39.0	degrees
Specific Gravity	2.55	

Reynolds No. = U*D50/v, where U=Shear Velocity, v=viscosity

U=(gRS)^0.5 for Smin [illegible]

Reynolds # for Smin [illegible]

$U=(gRS)^{0.5}$ for Smax

Reynolds # for Smax [illegible]

T = G*d*S where G=Unit weight of Water

Nb = F*T/(G(SD-1)D50)

F=(1/0.047)=21.3 for flat slopes with Reynolds No. < 500

F=(1/0.062)=16.1 for 500 < Reynolds No. < 40,000

F=varies from (1/0.062)=16.1 for Reynolds No. = 40,000 to (1/0.25)=4 for Reynolds No. = 500,000 or larger

K for S min (Compare K vs. R Chart)

K for S max (Compare K vs. R Chart)

F for S min

F for S max

SFh = (Cos a tan b)/(sin a + Nh tan b)

Tmax= K*G*d*S

Set K=0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope

K: 0.80

Ns = F*Tmax/(G(SG-1)D)

A = Atan(1/m)

B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar))+Sin(Ar))

Nsp = Ns(1+Sin(Ar+B)/2)

SFs = Cos(A)Tan(Ar)/(nTan(Ar)+Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	2.75	2.75	feet
T	[illegible]	12.08	lb/ft2
Nb	[illegible]	[illegible]	
Tmax	[illegible]	[illegible]	lb/ft2
Ns	[illegible]	[illegible]	
m Critical	[illegible]	[illegible]	
A (m crit)	[illegible]	[illegible]	degrees
B	[illegible]	[illegible]	degrees
Nsp	[illegible]	[illegible]	
SFh	[illegible]	[illegible]	
SFs	[illegible]	[illegible]	

HANSEN ALLEN & LUCE inc

Client *Wasatch Regional* | Sheet *1 of 2*
Project *Landfill Permit* | Comp. *GLJ*
Feature *Run-on Channel 1 G* | Chck'd *KCS*
Project # *113.30.100* | Date *17-Dec-04*

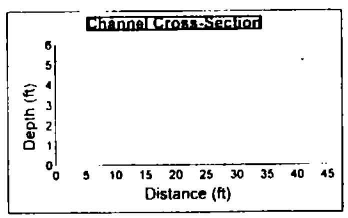
Trapezoidal Channel Flow Calculations

GENERAL CRITERIA:

	Design Flow:	551.00	cfs
	Bottom Width:	15.0	feet
	Side Slope1:	2.5	m1
	Side Slope2:	2.5	m2
	Friction Factor:		
	Assumed D50:	1.17	
Anderson et al. (1970)	If X=1, $n=0.0395(D50)^{1/6}$		
Abt. et al. (1987, 1988)	If X=2, $n=0.0456(D50*S)^{0.159}$		
	If X=3, $n=\{D50^{1/6}*(R/D50)^{1/6}\}/\{3.82*[2.25+5.23*LOG(R/D50)]\}$		
	Generally Applicable for R/D50 > 0.5		
Jarrett (1984)	If X=4, $n=0.39*(S^{0.38})*(R^{-0.16})$		
	If X=5, n=input n value		
	X:	1.0	
	Input n Value when X=5:		
	Min. Bottom Slope:	0.009	ft/ft
	Max. Bottom Slope:	0.011	ft/ft
	Freeboard:	1.0	feet
Depth Check	Depth (Min. Slope):	4.2	feet
	$Q-1.49AR^{(2/3)}S^{(1/2)}/n=$	[illegible]	Accuracy
	Calc (used) n Value:	[illegible]	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius		feet
	Velocity:		ft/sec
	Froude Number:		
Velocity Check	Depth (Max. Slope):	1.1	feet
	$Q-1.49AR^{(2/3)}S^{(1/2)}/n=$	[illegible]	Accuracy
	Calc (used) n Value:	[illegible]	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity:	[illegible]	ft/sec
	Froude Number:	[illegible]	

Channel Design Summary:

Bottom Width:		feet
Side Slope1:		1/m1
Side Slope2:		1/m2
Min. Bottom Slope:		ft/ft
Max. Bottom Slope:		ft/ft
Min. Channel Depth:		feet
Channel Top Width:		feet



HANSEN ALLEN & LUCE inc

Client *Wasatch Regional* | Sheet [illegible]
Project *Landfill Permit* | Comp *GLJ*
Feature *Run-on Channel 1-G* | Chck'd *KCS*
Project # *113.30.100* | Date *17-Dec-04*

DESIGN CRITERIA:

Design Flow:	[illegible]		cfs
Bottom Width:	[illegible]		feet
Side Slope1:	[illegible]		1/m1
Side Slope2:	[illegible]		1/m2
Friction Factor (Min. S & Max. S):	[illegible]	[illegible]	
Min. Bottom Slope:	[illegible]	%	
Max. Bottom Slope:	[illegible]	%	
Flow Depth (Min. S):	[illegible]		feet
Flow Depth (Max. S):	[illegible]		feet
Angle Repose (Ar):	39.0		degrees
Specific Gravity	2.55		

Reynolds No. = U*D50/v, where U=Shear Velocity, v=viscosity

U=(gRS)^0.5 for Smin	[illegible]
Reynolds # for Smin	[illegible]
$U=(gRS)^{0.5}$ for Smax	[illegible]
Reynolds # for Smax	[illegible]

T = G*d*S where G=Unit weight of Water
Nb = F*T/(G(SD-1)D50)
F=(1/0.047)=21.3 for flat slopes with Reynolds No. < 500
F=(1/0.062)=16.1 for 500 < Reynolds No. < 40,000
F=varies from (1/0.062)=16.1 for Reynolds No. = 40,000 to (1/0.25)=4 for Reynolds No. = 500,000 or larger
K for S min (Compare K vs. R Chart)
K for S max (Compare K vs. R Chart)
F for S min
F for S max
SFb = (Cos a tan b)/(sin a + Nb tan b)
Tmax = K*G*d*S
Set K=0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope
K | 0.80
Ns = 1*Tmax/(G(SG-1)D)
A = Atan(1/m)
B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar))+Sin(Ar))
Nsp = Ns(1+Sin(Ar+B)/2)
SFs = Cos(A)Tan(Ar)/(nTan(Ar)+Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	1.17	1.17	feet
T	[illegible]	[illegible]	lb/ft2
Nb	[illegible]	[illegible]	
Tmax	[illegible]	[illegible]	lb/ft2
Ns	[illegible]	[illegible]	
m Critical	[illegible]	[illegible]	
A (m crit)	[illegible]	[illegible]	degrees
B	[illegible]	[illegible]	degrees
Nsp	[illegible]	[illegible]	
SFb	[illegible]	[illegible]	
SFs	[illegible]	[illegible]	

HANSEN ALLEN & LUCE inc

Client	*Wasatch Regional*	Sheet	*1 of 2*
Project	*Landfill Permit*	Comp.	*GLJ*
Feature	*Run-on Channel 2-A*	Chck'd	*KCS*
Project #	*113.30.100*	Date	*17-Dec-04*

Trapezoidal Channel Flow Calculations

GENERAL CRITERIA:

Design Flow:	63.00	cfs
Bottom Width:	15.0	feet
Side Slope1:	2.5	m1
Side Slope2:	4.0	m2
Friction Factor:		
Assumed D50:	0.25	

Anderson et al. (1970) If X = 1, $n = 0.0395(D50)^{1/6}$

Abt et al. (1987, 1988) If X = 2, $n = 0.0456(D50*S)^{0.159}$

If X = 3, $n = \{D50^{1/6}*(R/D50)^{1/6}\}/\{3.82*[2.25+5.23*LOG(R/D50)]\}$

Generally Applicable for R/D50 > 0.5

Jarrett (1984) If X = 4, $n = 0.39*(S^{0.38})*(R^{-0.16})$

If X = 5, n = input n value

X:	1.0	
Input n Value when X = 5:		
Min. Bottom Slope:	0.002	ft/ft
Max. Bottom Slope:	0.003	ft/ft
Freeboard:	1.0	feet

Depth Check

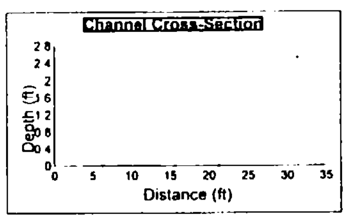
Depth (Min. Slope):	1.5	feet
$Q-1.49AR^{(2/3)}S^{(1/2)}/n=$	[illegible]	Accuracy
Calc (used) n Value:	[illegible]	
Required Depth:	[illegible]	feet
Area:	[illegible]	ft2
Perimeter:	[illegible]	feet
Hydraulic Radius:	[illegible]	feet
Velocity:	[illegible]	ft/sec
Froude Number:	[illegible]	

Velocity Check

Depth (Max. Slope):	1.5	feet
$Q-1.49AR^{(2/3)}S^{(1/2)}/n=$	0.000	Accuracy
Calc (used) n Value:	0.012	
Required Depth:	[illegible]	feet
Area:	[illegible]	ft2
Perimeter:	[illegible]	feet
Hydraulic Radius:	[illegible]	feet
Velocity:	[illegible]	ft/sec
Froude Number:	[illegible]	

Channel Design Summary:

Bottom Width:	feet
Side Slope1:	1/m1
Side Slope2:	1/m2
Min. Bottom Slope:	ft/ft
Max. Bottom Slope:	ft/ft
Min. Channel Depth:	feet
Channel Top Width:	feet



HANSEN ALLEN & LUCE inc ENGINEERS

Client	*Wasatch Regional*	Sheet	*2 of 2*
Project	*Landfill Permit*	Comp.	*GLJ*
Feature	*Run-on Channel 2-A*	Chck'd	*KCS*
Project #	*113.30.100*	Date	*17-Dec-04*

DESIGN CRITERIA:

Design Flow:	[illegible]	cfs
Bottom Width:	[illegible]	feet
Side Slope1:	[illegible]	1/m1
Side Slope2:	[illegible]	1/m2
Friction Factor (Min. S & Max. S):	[illegible]	[illegible]
Min. Bottom Slope:	[illegible]	%
Max. Bottom Slope:	[illegible]	%
Flow Depth (Min. S):	[illegible]	feet
Flow Depth (Max. S):	[illegible]	feet
Angle Repose (Ar):	39.0	degrees
Specific Gravity	2.55	

Reynolds No. = U*D50/v, where U = Shear Velocity, v = viscosity

U = (gRS)^0.5 for Smin

Reynolds # for Smin

$U = (gRS)^{0.5}$ for Smax

Reynolds # for Smax

T = G*d*S where G = Unit weight of Water

Nb = 1*1/(G(SD-1)D50)

F = (1/0.047) = 21.3 for flat slopes with Reynolds No. < 500

F = (1/0.062) = 16.1 for 500 < Reynolds No < 40,000

F = varies from (1/0.062) = 16.1 for Reynolds No. = 40,000 to (1/0.25) = 4 for Reynolds No. = 500,000 or larger

K for S min (Compare K vs. R Chart)

K for S max (Compare K vs. R Chart)

F for S min

F for S max

SFb = (Cos a tan b)/(sin a + Nb tan b)

Tmax = K*G*d*S

Set K = 0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope

K: 0.80

Ns = F*Tmax/(G(SG-1)D)

A = Atan(1/m)

B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar)) + Sin(Ar))

Nsp = Ns(1 + Sin(Ar + B)/2)

SFs = Cos(A)Tan(Ar)/(nTan(Ar) + Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	0.25	0.25	feet
T	[illegible]	[illegible]	lb/ft2
Nb	[illegible]	[illegible]	
Tmax	[illegible]	[illegible]	lb/ft2
Ns	[illegible]	[illegible]	
m Critical	[illegible]	[illegible]	
A (m crit)	[illegible]	[illegible]	degrees
B	[illegible]	[illegible]	degrees
Nsp	[illegible]	[illegible]	
SFb	[illegible]		
SFs	[illegible]		

HANSEN ALLEN & LUCE inc	Client *Wasatch Regional*	Sheet	*1 of 2*
	Project *Landfill Permit*	Comp.	*GLJ*
	Feature *Run-on Channel 2-B*	Chck'd	*KCS*
	Project # *113.30.100*	Date	*17-Dec-04*

Trapezoidal Channel Flow Calculations

GENERAL CRITERIA:

Design Flow:	86.00	cfs
Bottom Width:	15.0	feet
Side Slope1:	2.5	m1
Side Slope2:	4.0	m2
Friction Factor:		
Assumed D50:	1.00	

Anderson et al (1970) If X = 1, n = $0.0395(D50)^{1/6}$

Abt et al (1987, 1988) If X = 2, n = $0.0456(D50*S)^{0.159}$

If X = 3, n = $\{D50^{1/6}*(R/D50)^{1/6}\}/\{3.82*[2.25+5.23*LOG(R/D50)]\}$

Generally Applicable for R/D50 > 0.5

Jarrett (1984) If X = 4, n = $0.39*(S^{0.38})*(R^{-0.16})$

If X = 5, n = input n value

X	1.0	
Input n Value when X = 5:		
Min. Bottom Slope:	0.019	ft/ft
Max. Bottom Slope:	0.021	ft/ft
Freeboard:	1.0	feet

Depth Check

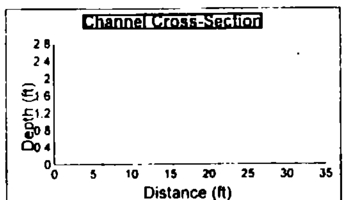
Depth (Min. Slope):	1.5	feet
$Q - 1.49AR^{(2/3)}S^{(1/2)}/n$ =	[illegible]	Accuracy
Calc (used) n Value:	[illegible]	
Required Depth:		feet
Area:		ft2
Perimeter		feet
Hydraulic Radius		feet
Velocity:		ft/sec
Froude Number:		

Velocity Check

Depth (Max. Slope):	1.5	feet
$Q - 1.49AR^{(2/3)}S^{(1/2)}/n$ =	[illegible]	Accuracy
Calc (used) n Value:	[illegible]	
Required Depth:	[illegible]	feet
Area:	[illegible]	ft2
Perimeter:	[illegible]	feet
Hydraulic Radius:	[illegible]	feet
Velocity:	[illegible]	ft/sec
Froude Number:	[illegible]	

Channel Design Summary:

Bottom Width	feet
Side Slope1:	1/m1
Side Slope2:	1/m2
Min. Bottom Slope:	ft/ft
Max. Bottom Slope:	ft/ft
Min. Channel Depth:	feet
Channel Top Width:	feet



HANSEN ALLEN & LUCE inc	Client *Wasatch Regional*	Sheet	*2 of 2*
	Project *Landfill Permit*	Comp.	*GLJ*
	Feature *Run-on Channel 2-B*	Chck'd	*KCS*
	Project # *113.30.100*	Date	*17-Dec-04*

DESIGN CRITERIA:

Design Flow:	[illegible]	cfs	
Bottom Width.	[illegible]	feet	
Side Slope1:	[illegible]	1/m1	
Side Slope2:	[illegible]	1/m2	
Friction Factor (Min. S & Max. S):	[illegible]	[illegible]	
Min. Bottom Slope:	[illegible]	%	
Max. Bottom Slope:	[illegible]	%	
Flow Depth (Min. S):	[illegible]	feet	
Flow Depth (Max. S):	[illegible]	feet	
Angle Repose (Ar):	39.0	degrees	
Specific Gravity	2.55		

Reynolds No. = U*D50/v, where U = Shear Velocity, v = viscosity

U = (gRS)^0.5 for Smin	[illegible]
Reynolds # for Smin	[illegible]
$U = (gRS)^{0.5}$ for Smax	[illegible]
Reynolds # for Smax	[illegible]

T = G*d*S where G = Unit weight of Water

Nb = F*T/(G(SD-1)D50)

F = (1/0.047) = 21.3 for flat slopes with Reynolds No. < 500

F = (1/0.062) = 16.1 for 500 < Reynolds No. < 40,000

F = varies from (1/0.062) = 16.1 for Reynolds No. = 40,000 to (1/0.25) = 4 for Reynolds No. = 500,000 or larger

K for S min (Compare K vs. R Chart) [illegible]

K for S max (Compare K vs. R Chart)

F for S min

F for S max

SFb = (Cos a tan b)/(sin a + Nb tan b)

Tmax = K*G*d*S

Set K = 0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope

K 0.80

Ns = F*Tmax/(G(SG-1)D)

A = Atan(1/m)

B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar)) + Sin(Ar))

Nsp = Ns(1 + Sin(Ar + B)/2)

SFs = Cos(A)Tan(Ar)/(nTan(Ar) + Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	1.00	1.00	feet
T	[illegible]	[illegible]	lb/ft2
Nb	[illegible]	[illegible]	
Tmax	[illegible]	[illegible]	lb/ft2
Ns	[illegible]	[illegible]	
m Critical	[illegible]	[illegible]	
A (m crit)	[illegible]	[illegible]	degrees
B	[illegible]	[illegible]	degrees
Nsp	[illegible]	[illegible]	
SFb	[illegible]	[illegible]	
SFs	[illegible]	[illegible]	

HANSEN ALLEN & LUCE inc ENGINEERS	
Client	Wasatch Regional
Project	Landfill Permit
Feature	Run-on Channel 2-C
Project #	113.30.100
Sheet	1 of 2
Comp.	GLJ
Chck'd	KCS
Date	17-Dec-04

Trapezoidal Channel Flow Calculations

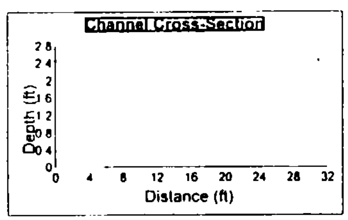
GENERAL CRITERIA:

	Design Flow:	86.00	cfs
	Bottom Width:	15.0	feet
	Side Slope1:	2.5	m1
	Side Slope2:	4.0	m2
	Friction Factor:		
	Assumed D50:	1.75	
Anderson et al (1970)	If X = 1, n = 0.0395(D50)^1/6		
Abt et al (1987, 1988)	If X = 2, n = 0.0456(D50*S)^0.159		
	If X = 3, n = {D50^1/6*(R/D50)^1/6}/{3.82*[2.25+5.23*LOG(R/D50)]}		
	Generally Applicable for R/D50 > 0.5		
Jarrett (1984)	If X = 4, n = 0.39*(S^0.38)*(R^-0.16)		
	If X = 5, n = input n value		
	X	4.0	
	Input n Value when X = 5:		
	Min. Bottom Slope:	0.019	ft/ft
	Max. Bottom Slope:	0.051	ft/ft
	Freeboard:	1.0	feet

Depth Check	Depth (Min. Slope):	1.5	feet
	$Q-1.49AR^{(2/3)}S^{(1/2)}/n=$	[illegible]	Accuracy
	Calc (used) n Value:	[illegible]	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity:	[illegible]	ft/sec
	Froude Number:	[illegible]	
Velocity Check	Depth (Max. Slope):	1.5	feet
	$Q-1.49AR^{(2/3)}S^{(1/2)}/n=$	[illegible]	Accuracy
	Calc (used) n Value:	[illegible]	
	Required Depth:	[illegible]	feet
	Area:	[illegible]	ft2
	Perimeter:	[illegible]	feet
	Hydraulic Radius:	[illegible]	feet
	Velocity:	[illegible]	ft/sec
	Froude Number:	[illegible]	

Channel Design Summary:

Bottom Width		feet
Side Slope1		1/m1
Side Slope2		1/m2
Min. Bottom Slope		ft/ft
Max. Bottom Slope		ft/ft
Min. Channel Depth		feet
Channel Top Width		feet



HANSEN ALLEN & LUCE inc ENGINEERS	
Client	Wasatch Regional
Project	Landfill Permit
Feature	Run-on Channel 2-C
Project #	113.30.100
Sheet	2 of 2
Comp	GLJ
Chck'd	KCS
Date	17-Dec-04

DESIGN CRITERIA:

Design Flow:	[illegible]	cfs
Bottom Width:	[illegible]	feet
Side Slope1:	[illegible]	1/m1
Side Slope2:	[illegible]	1/m2
Friction Factor (Min. S & Max. S):	[illegible]	
Min. Bottom Slope:	[illegible]	%
Max. Bottom Slope:	[illegible]	%
Flow Depth (Min. S):	[illegible]	feet
Flow Depth (Max. S):	[illegible]	feet
Angle Repose (Ar):	39.0	degrees
Specific Gravity	2.55	

Reynolds No. = U*D50/v, where U = Shear Velocity, v = viscosity

U = (gRS)^0.5 for Smin

Reynolds # for Smin

U = (gRS)^0.5 for Smax

Reynolds # for Smax

T = G*d*S where G = Unit weight of Water

Nh = F*T/(G(SD-1)D50)

F = (1/0.047) = 21.3 for flat slopes with Reynolds No. < 500

F = (1/0.062) = 16.1 for 500 < Reynolds No. < 40,000

F = varies from (1/0.062) = 16.1 for Reynolds No. = 40,000 to (1/0.25) = 4 for Reynolds No. = 500,000 or larger

K for S min (Compare K vs. R Chart)	[illegible]
K for S max (Compare K vs. R Chart)	[illegible]
F for S min	[illegible]
F for S max	[illegible]

SFb = (Cos a tan b)/(sin a + Nb tan b)

Tmax = K*G*d*S

Set K = 0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope

K: 0.80

Ns = F*Tmax/(G(SG-1)D)

A = Atan(1/m)

B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar)) + Sin(Ar))

Nsp = Ns(1 + Sin(Ar + B)/2)

SFs = Cos(A)Tan(Ar)/(nTan(Ar) + Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	1.75	1.75	feet
T	[illegible]	[illegible]	lb/ft2
Nh	[illegible]	[illegible]	
Tmax	[illegible]	[illegible]	lb/ft2
Ns	[illegible]	[illegible]	
m Critical	[illegible]	[illegible]	
A (m crit)	[illegible]	[illegible]	degrees
B	[illegible]	[illegible]	degrees
Nsp	[illegible]	[illegible]	
SFb	[illegible]		
SFs			

HANSEN ALLEN & LUCE inc	
Client	Wasatch Regional
Project	Landfill Permit
Feature	Run-on Channel 2-D
Project #	113.30.100
Sheet	1 of 2
Comp.	GLJ
Chck'd	KCS
Date	17-Dec-04

Trapezoidal Channel Flow Calculations

GENERAL CRITERIA:

Design Flow:	86.00	cfs
Bottom Width:	15.0	feet
Side Slope1:	2.5	m1
Side Slope2:	4.0	m2
Friction Factor:		
Assumed D50:	2.50	

Anderson et al. (1970) If X = 1, n = 0.0395(D50)^1/6

Abt et al. (1987, 1988) If X = 2, n = 0.0456(D50*S)^0.159

If X = 3, n = {D50^1/6*(R/D50)^1/6}/{3.82*[2.25+5.23*LOG(R/D50)]}

Generally Applicable for R/D50 > 0.5

Jarrett (1984) If X = 4, n = 0.39*(S^0.38)*(R^-0.16)

If X = 5, n = input n value

X:	4.0	
Input n Value when X = 5:		
Min. Bottom Slope:	0.149	ft/ft
Max. Bottom Slope:	0.151	ft/ft
Freeboard:	1.0	feet

Depth Check

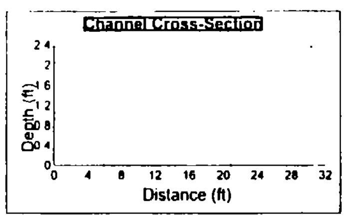
Depth (Min. Slope):	1.4	feet
Q-1.49AR^(2/3)S^(1/2)/n =	[illegible]	Accuracy
Calc (used) n Value:	[illegible]	
Required Depth:	[illegible]	feet
Area:	[illegible]	ft2
Perimeter:	[illegible]	feet
Hydraulic Radius:	[illegible]	feet
Velocity:	[illegible]	ft/sec
Froude Number:	[illegible]	

Velocity Check

Depth (Max. Slope):	1.4	feet
Q-1.49AR^(2/3)S^(1/2)/n =	[illegible]	Accuracy
Calc (used) n Value:	[illegible]	
Required Depth:	[illegible]	feet
Area:	[illegible]	ft2
Perimeter:	[illegible]	feet
Hydraulic Radius:	[illegible]	feet
Velocity:	[illegible]	ft/sec
Froude Number:	[illegible]	

Channel Design Summary:

Bottom Width		feet
Side Slope1		1/m1
Side Slope2		1/m2
Min. Bottom Slope:		ft/ft
Max. Bottom Slope:		ft/ft
Min. Channel Depth		feet
Channel Top Width:		feet



HANSEN ALLEN & LUCE inc	
Client	Wasatch Regional
Project	Landfill Permit
Feature	Run-on Channel 2-D
Project #	113.30.100
Sheet	2 of 2
Comp.	GLJ
Chck'd	KCS
Date	17-Dec-04

DESIGN CRITERIA:

Design Flow:		cfs
Bottom Width:		feet
Side Slope1		1/m1
Side Slope2		1/m2
Friction Factor (Min. S & Max. S):		
Min. Bottom Slope		%
Max. Bottom Slope		%
Flow Depth (Min. S).		feet
Flow Depth (Max. S):		feet
Angle Repose (Ar):	39.0	degrees
Specific Gravity	2.55	

Reynolds No. = U*D50/v, where U = Shear Velocity, v = viscosity

U = (gRS)^0.5 for Smin

Reynolds # for Smin

U = (gRS)^0.5 for Smax

Reynolds # for Smax

T = G*d*S where G = Unit weight of Water

Nb = F*T/(G(SD-1)D50)

F = (1/0.047) = 21.3 for flat slopes with Reynolds No. < 500

F = (1/0.062) = 16.1 for 500 < Reynolds No. < 40,000

F = varies from (1/0.062) = 16.1 for Reynolds No. = 40,000 to (1/0.25) = 4 for Reynolds No. = 500,000 or larger

K for S min (Compare K vs. R Chart) 0.16

K for S max (Compare K vs. R Chart) 0.16

F for S min

F for S max

SFb = (Cos a tan b)/(sin a + Nb tan b)

Tmax = K*G*d*S

Set K = 0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope

K: 0.80

Ns = F*Tmax/(G(SG-1)D)

A = Atan(1/m)

B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar)) + Sin(Ar))

Nsp = Ns(1 + Sin(Ar + B)/2)

SFs = Cos(A)Tan(Ar)/(nTan(Ar) + Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	2.50	2.50	feet
T			lb/ft2
Nb			
Tmax			lb/ft2
Ns			
m Critical			
A (m crit)			degrees
B			degrees
Nsp	[illegible]	[illegible]	
SFb	[illegible]	[illegible]	
SFs	[illegible]	[illegible]	

HANSEN ALLEN & LUCE INC ENGINEERS

Client	*Wasatch Regional*	Sheet	*1 of 2*
Project	*Landfill Permit*	Comp.	*GLJ*
Feature	*Run-on Channel 2-E*	Chck'd	*KCS*
Project #	*113.30.100*	Date	*17-Dec-04*

Trapezoidal Channel Flow Calculations

GENERAL CRITERIA:

Design Flow:	56.00	cfs
Bottom Width:	15.0	feet
Side Slope1:	2.5	m1
Side Slope2:	2.5	m2
Friction Factor:		
Assumed D50:	0.75	

Anderson et al (1970) If X = 1, n = 0.0395(D50)$^{1/6}$

Abt et al (1987, 1988) If X = 2, n = 0.0456(D50*S)$^{0.159}$

If X = 3, n = {D50$^{1/6}$*(R/D50)$^{1/6}$}/{3.82*[2.25 + 5.23*LOG(R/D50)]}

Generally Applicable for R/D50 > 0.5

Jarrett (1984) If X = 4, n = 0.39*(S$^{0.38}$)*(R$^{0.16}$)

If X = 5, n = input n value

X:	1.0	
Input n Value when X = 5:		
Min. Bottom Slope:	0.015	ft/ft
Max. Bottom Slope:	0.015	ft/ft
Freeboard	1.0	feet

Depth Check

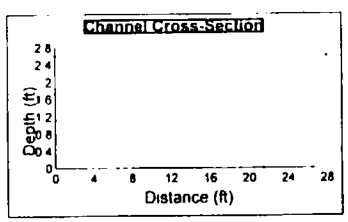
Depth (Min. Slope):	1.6	feet
Q-1.49AR$^{(2/3)}$S$^{(1/2)}$/n =		Accuracy
Calc (used) n Value:		
Required Depth:		feet
Area:		ft2
Perimeter:		feet
Hydraulic Radius:		feet
Velocity:		ft/sec
Froude Number:		

Velocity Check

Depth (Max. Slope):	1.6	feet
Q-1.49AR$^{(2/3)}$S$^{(1/2)}$/n =		Accuracy
Calc (used) n Value:		
Required Depth:		feet
Area:		ft2
Perimeter:		feet
Hydraulic Radius:		feet
Velocity:		ft/sec
Froude Number:		

Channel Design Summary:

Bottom Width:	feet
Side Slope1:	1/m1
Side Slope2:	1/m2
Min. Bottom Slope:	ft/ft
Max. Bottom Slope:	ft/ft
Min. Channel Depth:	feet
Channel Top Width:	feet



HANSEN ALLEN & LUCE INC ENGINEERS

Client	*Wasatch Regional*	Sheet	
Project	*Landfill Permit*	Comp.	*GLJ*
Feature	*Run-on Channel 2-E*	Chck'd	*KCS*
Project #	*113.30.100*	Date	*17-Dec-04*

DESIGN CRITERIA:

Design Flow:		cfs
Bottom Width:		feet
Side Slope1:		1/m1
Side Slope2:		1/m2
Friction Factor (Min. S & Max. S):		
Min. Bottom Slope:		%
Max. Bottom Slope:		%
Flow Depth (Min. S):		feet
Flow Depth (Max. S):		feet
Angle Repose (Ar):	39.0	degrees
Specific Gravity	2.55	

Reynolds No. = U*D50/v, where U = Shear Velocity, v = viscosity

U = (gRS)^0.5 for Smin

Reynolds # for Smin

U = (gRS)$^{0.5}$ for Smax

Reynolds # for Smax

T = G*d*S where G = Unit weight of Water

Nb = F*T/(G(SD-1)D50)

F = (1/0.047) = 21.3 for flat slopes with Reynolds No. < 500

F = (1/0.062) = 16.1 for 500 < Reynolds No. < 40,000

F = varies from (1/0.062) = 16.1 for Reynolds No. = 40,000 to (1/0.25) = 4 for Reynolds No. = 500,000 or larger

K for S min (Compare K vs. R Chart)

K for S max (Compare K vs. R Chart)

F for S min

F for S max

SFb = (Cos a tan b)/(sin a + Nb tan b)

Tmax = K*G*d*S

Set K = 0.75 for 1.5:1 slope, 0.76 for 2:1 slope, and 0.85 for 3:1 slope

K: 0.80

Ns = F*Tmax/(G(SG-1)D)

A = Atan(1/m)

B = Atan(Cos(Ar)/(2Sin(A)/NsTan(Ar)) + Sin(Ar))

Nsp = Ns(1 + Sin(Ar + B)/2)

SFs = Cos(A)Tan(Ar)/(nTan(Ar) + Sin(A)Cos(B))

RIPRAP DESIGN:

	Smin	Smax	
D50	0.75	0.75	feet
T			lb/ft2
Nb			
Tmax			lb/ft2
Ns			
m Critical			
A (m crit)			degrees
B			degrees
Nsp			
SFb			
SFs			

CLOSURE HYDROLO

Purpose: To determine the runoff from the closure cap of the Wasatch Regional Facility.

Method: The SCS curve number method was used with the HEC-1 hydrology model. The HEC-1 model was set up using the HAL Water Suite.

Required: In order to calculate the runoff the following steps and information are required:

- A delineation of the tributary area.
- A representative Soil Conservation Service(SCS) curve number (CN) for the tributary area.
- Lag time.
- Storm Distribution.
- 100 year-24 hour precipitation.

Delineation: The delineation of the subbasins, shown in Figure 1, was based on the preliminary cell closure cap design. Each basin would drain into a channel which would convey the runoff to a down spout that would take the water off of the cell.

Curve Numbers: The curve numbers were determined based on the hydrologic soil type, Type B, found in the area because native soils are going to be used for cover. The cover type was assumed to be similar to a dirt road. The cover conditions were combined with the hydrologic soil type to produce a curve number based on Table 2-2a of Technical Release 55. A curve number of 82 was applied to all subbasins.

Precipitation: A 100 year - 24 hour event was used for the design storm. The rainfall amount was taken from the "Point Precipitation Frequency Estimates from NOAA Atlas 14". The value for a 100 year - 24 hour event was 2.52 inches.

Storm Distribution: The distribution used for the 24-hour event was the SCS Type II.

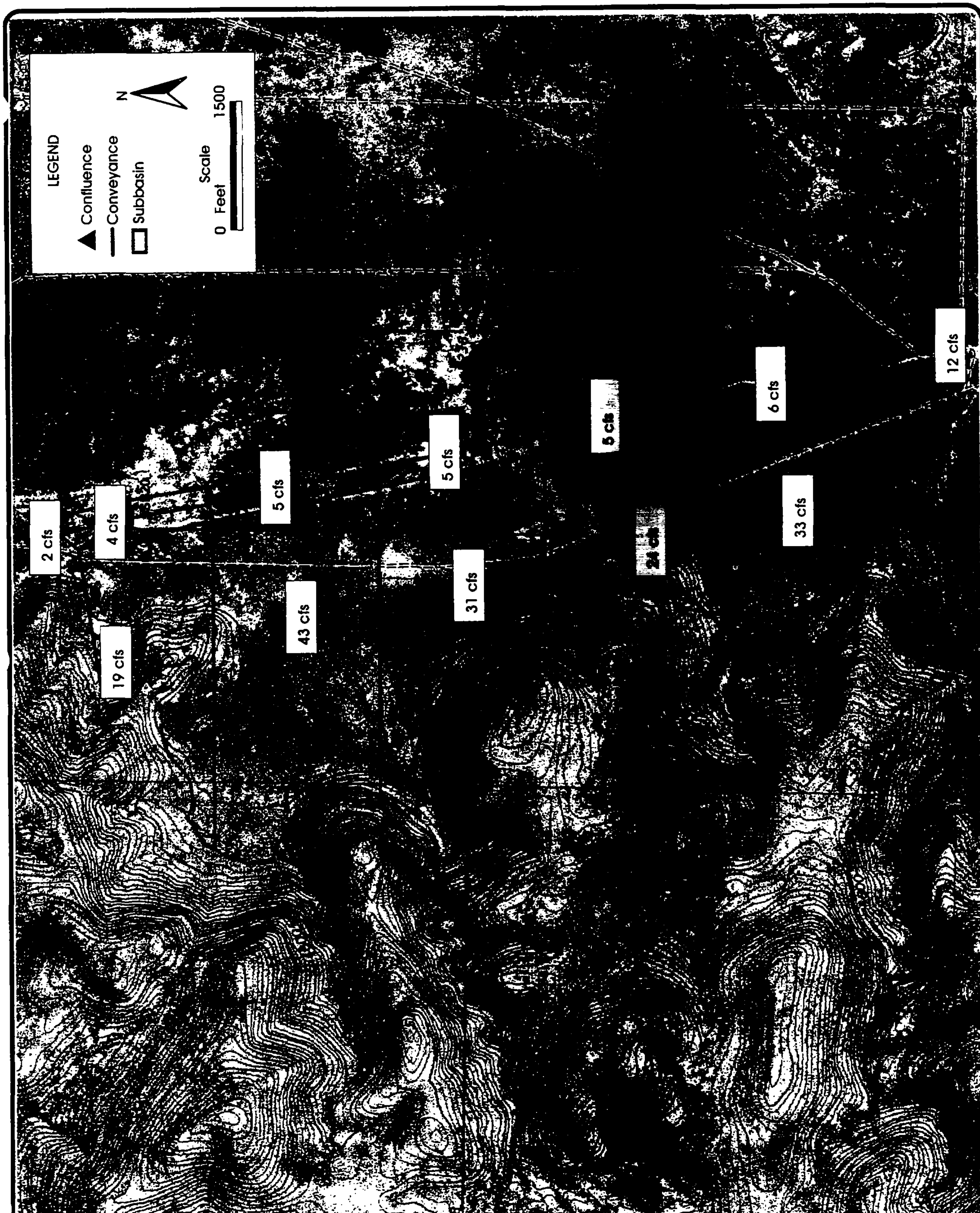
Lag Time: The lag times were calculated by using the Time of Concentration and the equation $T_L = 0.6Tc$. Tc was calculated using Worksheet 3 in TR-55. A calculation sheet for the subbasins is provided and are labeled with their subbasin name.

Results: The results of the HEC-1 model run are summarized in Figure 2 and can be found on page 25 of the HEC-1 output. The maximum runoff from the top of the cap was 41 cfs. The maximum runoff from the side slopes was 16 cfs.

HANSEN ALLEN & LUCE INC ENGINEERS

ON-SITE HYDROLOGY MODEL

FIGURE 1



HANSEN ALLEN & LUCE INC
ENGINEERS

ON-SITE MODEL RESULTS

FIGURE 2

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

TIME OF MAX STAGE	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE
+	HYDROGRAPH AT	SB7	19.	13.08	8.	3.	3	.11	
+	ROUTED TO	CV3	19.	13.08	8.	3.	3.	.11	
+	HYDROGRAPH AT	SB19	3.	13.00	1.	0.	0	.01	
+	2 COMBINED AT	HC3	21.	13.08	9.	3.	3	.12	
+	HYDROGRAPH AT	SB20	3.	13.00	1.	0.	0.	.01	
+	ROUTED TO	CV4	3.	13.00	1.	0.	0.	.01	
+	2 COMBINED AT	HC4	23.	13.00	10.	4.	4.	.14	
+	HYDROGRAPH AT	SB46	1.	13.00	1.	0.	0.	.01	
+	ROUTED TO	CV32	1.	13.00	1.	0.	0.	.01	
+	HYDROGRAPH AT	SB47	1.	13.00	0.	0.	0.	.01	
+	2 COMBINED AT	HC33	2.	13.00	1.	0.	0.	.01	
+	HYDROGRAPH AT	SB8	43.	13.08	19.	7.	7.	.25	
+	ROUTED TO	CV8	43.	13.08	19.	7.	7.	.25	
+	HYDROGRAPH AT	SB24	3.	13.00	1.	0.	0.	.02	
+	2 COMBINED AT	HC8	45.	13.08	20.	7.	7.	.27	
+	ROUTED TO	CV9	45.	13.08	20.	7.	7.	.27	
+	HYDROGRAPH AT	SB25	3.	13.00	1.	0.	0.	.01	
+	2 COMBINED AT	HC9	48.	13.08	21.	8.	8.	.28	
+	HYDROGRAPH AT	SB9	31.	13.08	13.	5.	5.	.18	
+	ROUTED TO	CV13	31.	13.08	13.	5.	5.	.18	
+	HYDROGRAPH AT	SB29	3.	13.00	1.	0.	0.	.02	
+	2 COMBINED AT	HC13	33.	13.08	15.	5.	5.	.20	
+	ROUTED TO	CV14	33.	13.08	15.	5.	5.	.20	
+	HYDROGRAPH AT	SB30	3	13.00	1.	0.	0.	.01	
+	2 COMBINED AT	HC14	36.	13.08	16.	6.	6	21	
+	HYDROGRAPH AT	SB10	24	13.08	10.	4.	4	.13	
+	ROUTED TO	CV18	24.	13.08	10.	4.	4.	.13	

+	HYDROGRAPH AT	SB34	3.	13.00	1.	0.	0.	02
+	2 COMBINED AT	HC18	27.	13.00	11.	4.	4.	.15
+	ROUTED TO	CV19	27.	13.00	11.	4.	4.	15
+	HYDROGRAPH AT	SB35	3.	13.00	1.	0.	0.	01
+	2 COMBINED AT	HC19	29.	13.00	12.	4.	4.	16
+	HYDROGRAPH AT	SB11	33.	13.08	14.	5.	5.	19
+	ROUTED TO	CV23	33.	13.08	14.	5.	5.	19
+	HYDROGRAPH AT	SB39	3.	13.00	1.	0.	0.	02
+	2 COMBINED AT	HC23	36.	13.00	15.	6.	6.	20
+	ROUTED TO	CV24	36.	13.00	15.	6.	6.	.20
+	HYDROGRAPH AT	SB40	3.	13.00	1.	0.	0.	.01
+	2 COMBINED AT	HC24	39.	13.00	16.	6.	6.	.22
+	HYDROGRAPH AT	SB51	8.	13.00	3.	1.	1.	.04
+	ROUTED TO	CV29	8.	13.00	3.	1.	1.	04
+	HYDROGRAPH AT	SB52	4.	13.00	2.	1.	1.	.02
+	2 COMBINED AT	HC29	12.	13.00	5.	2.	2.	06

```
1***************************************                                        ***************************************
*                                     *                                         *                                     *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1) *                                         *   U.S. ARMY CORPS OF ENGINEERS      *
*              JUN   1998             *                                         *   HYDROLOGIC ENGINEERING CENTER     *
*           VERSION 4.1               *                                         *        609 SECOND STREET            *
*                                     *                                         *     DAVIS, CALIFORNIA 95616         *
* RUN DATE   08DEC04  TIME  14:47:27  *                                         *        (916) 756-1104               *
*                                     *                                         *                                     *
***************************************                                         ***************************************


                                    X     X  XXXXXXX   XXXXX          X
                                    X     X  X        X     X        XX
                                    X     X  X        X               X
                                    XXXXXXX  XXXX     X       XXXXX   X
                                    X     X  X        X               X
                                    X     X  X        X     X         X
                                    X     X  XXXXXXX   XXXXX         XXX


            THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

            THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
            THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
            NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
            DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
            KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1                                                    HEC-1 INPUT

           LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

                         *DIAGRAM
 *** FREE ***
             1           ID    Closure C:\gisfiles\113\30.100\Closure\Closure.cnt
             2           ID
             3           ID
             4           IT     5                      288
             5           IO     3

             6           KK   SB7
             7           BA  .1112
             8           PB   2.52
             9           IN    30
            10           PI     0    .005    .006    .006    .006    .006    .006    .007    .007    .007
            11           PI  .008    .008    .009    .009     .01     .01     .01    .012    .015    .016
            12           PI  .018    .023    .033    .046    .038    .072    .037    .027    .023    .018
            13           PI  .015    .013    .012    .011    .011     .01    .009    .009    .008    .008
            14           PI  .008    .008    .006    .006    .006    .005    .005    .005    .005
            15           LS     0      82
            16           UD   .27
            17           KO                                  22

            18           KK   CV3
            19           RD 202.205 0.25000   0.030          CIRC    1.50    0.00
            20           KO                                  22

            21           KK  SB19
            22           BA  .0133
            23           LS     0      82
            24           UD   .08
            25           KO                                  22

            26           KK   HC3
            27           HC     2
            28           KO                                  22

            29           KK  SB20
            30           BA  .0134
            31           LS     0      82
            32           UD   .08
            33           KO                                  22

            34           KK   CV4
            35           RD 209.994 0.25000   0.030          CIRC    1.50    0.00
            36           KO                                  22

            37           KK   HC4
            38           HC     2
            39           KO                                  22
```

```
  40          KK    SB46
  41          BA   .0070
  42          LS       0       82
  43          UD     .08
  44          KO                                        22
1                                                HEC-1 INPUT

LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


  45          KK    CV32
  46          RD 254.932 0.25000   0.030            CIRC    1.50    0.00
  47          KO                                        22

  48          KK    SB47
  49          BA   .0060
  50          LS       0       82
  51          UD     .08
  52          KO                                        22

  53          KK    HC33
  54          HC       2
  55          KO                                        22

  56          KK     SB8
  57          BA   .2549
  58          LS       0       82
  59          UD     .27
  60          KO                                        22

  61          KK     CV8
  62          RD 203.352 0.25000   0.030            CIRC    1.50    0.00
  63          KO                                        22

  64          KK    SB24
  65          BA   .0157
  66          LS       0       82
  67          UD     .08
  68          KO                                        22

  69          KK     HC8
  70          HC       2
  71          KO                                        22

  72          KK     CV9
  73          RD 206.256 0.25000   0.030            CIRC    1.50    0.00
  74          KO                                        22

  75          KK    SB25
  76          BA   .0143
  77          LS       0       82
  78          UD     .08
  79          KO                                        22

  80          KK     HC9
  81          HC       2
  82          KO                                        22

  83          KK     SB9
  84          BA   .1828
  85          LS       0       82
  86          UD     .27
  87          KO                                        22
1                                                HEC-1 INPUT

LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


  88          KK    CV13
  89          RD 206.577 0.25000   0.030            CIRC    1.50    0.00
  90          KO                                        22

  91          KK    SB29
  92          BA   .0157
  93          LS       0       82
  94          UD     .08
  95          KO                                        22

  96          KK    HC13
  97          HC       2
  98          KO                                        22

  99          KK    CV14
 100          RD 201.741 0.25000   0.030            CIRC    1.50    0.00
 101          KO                                        22

 102          KK    SB30
```

```
103          BA    .0143
104          LS        0       82
105          UD      .08
106          KO                                        22

107          KK     HC14
108          HC        2
109          KO                                        22

110          KK     SB10
111          BA    .1345
112          LS        0       82
113          UD      .19
114          KO                                        22

115          KK     CV18
116          RD 204.965 0.25000   0.030             CIRC     1.50     0.00
117          KO                                        22

118          KK     SB34
119          BA    .0158
120          LS        0       82
121          UD      .08
122          KO                                        22

123          KK     HC18
124          HC        2
125          KO                                        22

126          KK     CV19
127          RD 206.577 0.25000   0.030             CIRC     1.50     0.00
128          KO                                        22
1                                             HEC-1 INPUT

LINE         ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


129          KK     SB35
130          BA    .0143
131          LS        0       82
132          UD      .08
133          KO                                        22

134          KK     HC19
135          HC        2
136          KO                                        22

137          KK     SB11
138          BA    .1875
139          LS        0       82
140          UD      .19
141          KO                                        22

142          KK     CV23
143          RD 206.577 0.25000   0.030             CIRC     1.50     0.00
144          KO                                        22

145          KK     SB39
146          BA    .0156
147          LS        0       82
148          UD      .08
149          KO                                        22

150          KK     HC23
151          HC        2
152          KO                                        22

153          KK     CV24
154          RD 202.083 0.25000   0.030             CIRC     1.50     0.00
155          KO                                        22

156          KK     SB40
157          BA    .0143
158          LS        0       82
159          UD      .08
160          KO                                        22

161          KK     HC24
162          HC        2
163          KO                                        22

164          KK     SB51
165          BA    .0420
166          LS        0       82
167          UD      .08
168          KO                                        22
                                              HEC-1 INPUT
```

```
LINE          ID.......1.......2.......3.......4.......5...  ...6... . .7.......8.......9......10

 169          KK    CV29
 170          RD 323.925 0.25000   0.030            CIRC     1.50     0.00
 171          KO                                      22

 172          KK    SB52
 173          BA   .0209
 174          LS       0       82
 175          UD     .08
 176          KO                                      22

 177          KK    HC29
 178          HC       2
 179          KO                                      22
 180          ZZ
1
              SCHEMATIC DIAGRAM OF STREAM NETWORK
INPUT
 LINE       (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

  NO.       (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

   6          SB7
               V
               V
  18          CV3
               .
               .
  21           .          SB19
               .             .
               .             .
  26          HC3............
               .
               .
  29           .          SB20
               .             V
               .             V
  34           .           CV4
               .             .
               .             .
  37          HC4............
               .
               .
  40           .          SB46
               .             V
               .             V
  45           .          CV32
               .             .
               .             .
  48           .             .           SB47
               .             .              .
               .             .              .
  53           .          HC33...............
               .             .
               .             .
  56           .             .            SB8
               .             .              V
               .             .              V
  61           .             .            CV8
               .             .              .
               .             .              .
  64           .             .              .          SB24
               .             .              .             .
               .             .              .             .
  69           .             .            HC8.............
               .             .              V
               .             .              V
  72           .             .            CV9
               .             .              .
               .             .              .
  75           .             .              .          SB25
               .             .              .             .
               .             .              .             .
  80           .             .            HC9.............
               .             .              .
               .             .              .
  83           .             .              .           SB9
               .             .              .             V
               .             .              .             V
  88           .             .              .          CV13
               .             .              .             .
               .             .              .             .
  91           .             .              .             .          SB29
               .             .              .             .             .
               .             .              .             .             .
```

```
  96           .           .           .        HC13............
               .           .           .           V
               .           .           .           V
  99           .           .           .        CV14
               .           .           .           .
               .           .           .           .
 102           .           .           .           .        SB30
               .           .           .           .           .
               .           .           .           .           .
 107           .           .           .        HC14............
               .           .           .           .
               .           .           .           .
 110           .           .           .           .        SB10
               .           .           .           .           V
               .           .           .           .           V
 115           .           .           .           .        CV18
               .           .           .           .           .
               .           .           .           .           .
 118           .           .           .           .           .        SB34
               .           .           .           .           .           .
               .           .           .           .           .           .
 123           .           .           .           .        HC18............
               .           .           .           .           V
               .           .           .           .           V
 126           .           .           .           .        CV19
               .           .           .           .           .
               .           .           .           .           .
 129           .           .           .           .           .        SB35
               .           .           .           .           .           .
               .           .           .           .           .           .
 134           .           .           .           .        HC19............
               .           .           .           .           .
               .           .           .           .           .
 137           .           .           .           .           .        SB11
               .           .           .           .           .           V
               .           .           .           .           .           V
 142           .           .           .           .           .        CV23
               .           .           .           .           .           .
               .           .           .           .           .           .
 145           .           .           .           .           .           .        SB39
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 150           .           .           .           .           .        HC23............
               .           .           .           .           .           V
               .           .           .           .           .           V
 153           .           .           .           .           .        CV24
               .           .           .           .           .           .
               .           .           .           .           .           .
 156           .           .           .           .           .           .        SB40
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 161           .           .           .           .           .        HC24............
               .           .           .           .           .           .
               .           .           .           .           .           .
 164           .           .           .           .           .           .        SB51
               .           .           .           .           .           .           V
               .           .           .           .           .           .           V
 169           .           .           .           .           .           .        CV29
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 172           .           .           .           .           .           .           .        SB52
               .           .           .           .           .           .           .           .
               .           .           .           .           .           .           .           .
 177           .           .           .           .           .           .        HC29............
(***) RUNOFF ALSO COMPUTED AT THIS LOCATION
```

```
1****************************************
*                                       *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *
*              JUN   1998               *
*            VERSION 4.1                *
*                                       *
*  RUN DATE   08DEC04   TIME  14:47:27  *
*                                       *
*****************************************
```

```
*****************************************
*                                       *
*   U.S. ARMY CORPS OF ENGINEERS        *
*   HYDROLOGIC ENGINEERING CENTER       *
*          609 SECOND STREET            *
*       DAVIS, CALIFORNIA 95616         *
*            (916) 756-1104             *
*                                       *
*****************************************
```

Closure C:\gisfiles\113\30.100\Closure\Closure.cnt

```
   5 IO          OUTPUT CONTROL VARIABLES
                       IPRNT          3  PRINT CONTROL
                       IPLOT          0  PLOT CONTROL
```

```
                QSCAL           0.  HYDROGRAPH PLOT SCALE

  IT          HYDROGRAPH TIME DATA
                 NMIN           5  MINUTES IN COMPUTATION INTERVAL
                IDATE      1    0  STARTING DATE
                ITIME        0000  STARTING TIME
                   NQ         288  NUMBER OF HYDROGRAPH ORDINATES
               NDDATE      1    0  ENDING DATE
               NDTIME        2355  ENDING TIME
                ICENT          19  CENTURY MARK

           COMPUTATION INTERVAL     .08 HOURS
                TOTAL TIME BASE   23.92 HOURS

       ENGLISH UNITS
            DRAINAGE AREA          SQUARE MILES
            PRECIPITATION DEPTH    INCHES
            LENGTH, ELEVATION      FEET
            FLOW                   CUBIC FEET PER SECOND
            STORAGE VOLUME         ACRE-FEET
            SURFACE AREA           ACRES
            TEMPERATURE            DEGREES FAHRENHEIT

*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

         **************
         *            *
  6 KK   *      SB7   *
         *            *
         **************

  9 IN        TIME DATA FOR INPUT TIME SERIES
                JXMIN          30  TIME INTERVAL IN MINUTES
               JXDATE      1    0  STARTING DATE
               JXTIME           0  STARTING TIME

 17 KO        OUTPUT CONTROL VARIABLES
                IPRNT           3  PRINT CONTROL
                IPLOT           0  PLOT CONTROL
                QSCAL          0.  HYDROGRAPH PLOT SCALE
                IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                 IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
               TIMINT        .083  TIME INTERVAL IN HOURS

            SUBBASIN RUNOFF DATA

  7 BA        SUBBASIN CHARACTERISTICS
                TAREA         .11  SUBBASIN AREA

              PRECIPITATION DATA

  8 PB          STORM        2.52  BASIN TOTAL PRECIPITATION

 10 PI          INCREMENTAL PRECIPITATION PATTERN
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .01      .01      .01      .01      .01      .01      .01      .01
                .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                .01      .01      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
```

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                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00

  15 LS         SCS LOSS RATE
                     STRTL         .44  INITIAL ABSTRACTION
                    CRVNBR       82.00  CURVE NUMBER
                     RTIMP         .00  PERCENT IMPERVIOUS AREA

  16 UD         SCS DIMENSIONLESS UNITGRAPH
                      TLAG         .27  LAG

                                                              ***

                                                         UNIT HYDROGRAPH
                                                    18 END-OF-PERIOD ORDINATES
               28.       92.      161.      171.      143.       96.       60.       40.       25.       17
               11.        7.        4.        3.        2.        1.        1.        0.

      ***             ***             ***             ***             ***

                            HYDROGRAPH AT STATION       SB7

     TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01

   PEAK FLOW     TIME                         MAXIMUM AVERAGE FLOW
                                       6-HR        24-HR       72-HR       23.92-HR
 +   (CFS)       (HR)
                            (CFS)
 +      19.     13.08                    8.           3.          3.            3.
                          (INCHES)     .680        1.002       1.002         1.002
                           (AC-FT)       4.           6.          6.            6.

                           CUMULATIVE AREA =      .11 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

             **************
             *            *
  18 KK      *      CV3   *
             *            *
             **************

  20 KO         OUTPUT CONTROL VARIABLES
                     IPRNT           3  PRINT CONTROL
                     IPLOT           0  PLOT CONTROL
                     QSCAL          0.  HYDROGRAPH PLOT SCALE
                     IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                      IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                     ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                     ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                    TIMINT        .083  TIME INTERVAL IN HOURS


              HYDROGRAPH ROUTING DATA

  19 RD         MUSKINGUM-CUNGE CHANNEL ROUTING
                         L        202.  CHANNEL LENGTH
                         S       .2500  SLOPE
                         N        .030  CHANNEL ROUGHNESS COEFFICIENT
                        CA         .00  CONTRIBUTING AREA
                     SHAPE        CIRC  CHANNEL SHAPE
                        WD        1.50  BOTTOM WIDTH OR DIAMETER
                         Z         .00  SIDE SLOPE

                                                              ***
                                   COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                         COMPUTATION TIME STEP
                  ELEMENT    ALPHA      M          DT         DX        PEAK     TIME TO     VOLUME      MAXIMUM
                                                                                  PEAK                   CELERITY
                                                  (MIN)      (FT)      (CFS)     (MIN)       (IN)         (FPS)

                    MAIN     14.34     1.25        .20     101.10      18.77     785.32      1.00        16.47

                                          INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                    MAIN     14.34     1.25       5.00                 18.75     785.00      1.00

CONTINUITY SUMMARY  AC FT  - INFLOW= .5945E+01 EXCESS= .0000E+00 OUTFLOW= .5944E-01 BASIN STORAGE= .9746E-03 PERCENT ERROR=    .0
```

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      ***              ***              ***              ***              ***

                          HYDROGRAPH AT STATION      CV3

  PEAK FLOW      TIME                               MAXIMUM AVERAGE FLOW
                                       6-HR        24-HR        72-HR        23.92-HR
+    (CFS)       (HR)
                            (CFS)
+       19.     13.08                    8.           3.           3.              3.
                          (INCHES)      680        1.002        1.002           1.002
                           (AC-FT)       4.           6.           6.              6.

                         CUMULATIVE AREA =      .11 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


            **************
            *            *
   21 KK    *     SB19   *
            *            *
            **************

   25 KO          OUTPUT CONTROL VARIABLES
                       IPRNT           3  PRINT CONTROL
                       IPLOT           0  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE
                       IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                        IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                       ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                       ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                      TIMINT        .083  TIME INTERVAL IN HOURS


                SUBBASIN RUNOFF DATA

   22 BA          SUBBASIN CHARACTERISTICS
                       TAREA         .01  SUBBASIN AREA

                  PRECIPITATION DATA

    8 PB               STORM        2.52  BASIN TOTAL PRECIPITATION

   10 PI            INCREMENTAL PRECIPITATION PATTERN
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .01      .01      .01      .01      .01      .01      .01      .01
                        .01      .01      .01      .01      .01      .01      .01      .01      .01       01
                        .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                        .01      .01      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00       00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                        .00      .00      .00      .00      .00      .00      .00      .00       00       00
                        .00      .00      .00      .00      .00      .00      .00

   23 LS          SCS LOSS RATE
                       STRTL         .44  INITIAL ABSTRACTION
                      CRVNBR       82.00  CURVE NUMBER
                       RTIMP         .00  PERCENT IMPERVIOUS AREA

   24 UD          SCS DIMENSIONLESS UNITGRAPH
                        TLAG         .08  LAG

                                                        ***
```

```
                                                       UNIT HYDROGRAPH
                                                   7 END-OF-PERIOD ORDINATES
           41.       42.       13.        4.        1.        1.        0.

     ***            ***            ***             ***              ***

                          HYDROGRAPH AT STATION     SB19

   TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01

  PEAK FLOW     TIME                              MAXIMUM AVERAGE FLOW
                                      6-HR       24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                           (CFS)
+      3.      13.00                    1.          0.          0.           0.
                         (INCHES)     .681       1.009       1.009       1.009
                          (AC-FT)       0.          1.          1.           1.

                          CUMULATIVE AREA =      .01 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


            **************
            *            *
   26 KK    *     HC3    *
            *            *
            **************

   28 KO          OUTPUT CONTROL VARIABLES
                       IPRNT            3  PRINT CONTROL
                       IPLOT            0  PLOT CONTROL
                       QSCAL           0.  HYDROGRAPH PLOT SCALE
                       IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                        IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                       ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                       ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                      TIMINT         .083  TIME INTERVAL IN HOURS


   27 HC          HYDROGRAPH COMBINATION
                       ICOMP            2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                               ***

     ***            ***            ***             ***              ***

                          HYDROGRAPH AT STATION      HC3

  PEAK FLOW     TIME                              MAXIMUM AVERAGE FLOW
                                      6-HR       24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                           (CFS)
+     21.      13.08                    9.          3.          3.           3.
                         (INCHES)     .680       1.003       1.003       1.003
                          (AC-FT)       5.          7.          7.           7.

                          CUMULATIVE AREA =      .12 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


            **************
            *            *
   29 KK    *    SB20    *
            *            *
            **************

   33 KO          OUTPUT CONTROL VARIABLES
                       IPRNT            3  PRINT CONTROL
                       IPLOT            0  PLOT CONTROL
                       QSCAL           0.  HYDROGRAPH PLOT SCALE
                       IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                        IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                       ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                       ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                      TIMINT          083  TIME INTERVAL IN HOURS

                  SUBBASIN RUNOFF DATA
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  30 BA        SUBBASIN CHARACTERISTICS
                     TAREA          .01  SUBBASIN AREA

               PRECIPITATION DATA

   8 PB              STORM         2.52  BASIN TOTAL PRECIPITATION

  10 PI          INCREMENTAL PRECIPITATION PATTERN
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .01       .01       .01       .01       .01       .01       .01       .01
                     .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                     .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                     .01       .01       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                     .00       .00       .00       .00       .00       .00       .00

  31 LS        SCS LOSS RATE
                     STRTL          .44  INITIAL ABSTRACTION
                    CRVNBR        82.00  CURVE NUMBER
                     RTIMP          .00  PERCENT IMPERVIOUS AREA

  32 UD        SCS DIMENSIONLESS UNITGRAPH
                      TLAG          .08  LAG

                                                          ***

                                                      UNIT HYDROGRAPH
                                                  7 END-OF-PERIOD ORDINATES
                42.        42.        14.         4.         1.         1.         0.

      ***            ***            ***            ***            ***

                       HYDROGRAPH AT STATION     SB20

    TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01

  PEAK FLOW     TIME                         MAXIMUM AVERAGE FLOW
                                      6-HR        24-HR        72-HR      23.92-HR
+   (CFS)       (HR)
                        (CFS)
+        3.    13.00                    1.           0.           0.            0.
                      (INCHES)        .681        1.009        1.009         1.009
                       (AC-FT)          0.           1.           1.            1.

                      CUMULATIVE AREA =      .01 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

            **************
            *            *
  34 KK     *     CV4    *
            *            *
            **************

  36 KO        OUTPUT CONTROL VARIABLES
                     IPRNT            3  PRINT CONTROL
                     IPLOT            0  PLOT CONTROL
                     QSCAL           0.  HYDROGRAPH PLOT SCALE
                     IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                      IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                     ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
```

```
                ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
               TIMINT         .083  TIME INTERVAL IN HOURS


          HYDROGRAPH ROUTING DATA

 35 RD       MUSKINGUM-CUNGE CHANNEL ROUTING
                    L         210.  CHANNEL LENGTH
                    S        .2500  SLOPE
                    N         .030  CHANNEL ROUGHNESS COEFFICIENT
                   CA          .00  CONTRIBUTING AREA
                SHAPE         CIRC  CHANNEL SHAPE
                   WD         1.50  BOTTOM WIDTH OR DIAMETER
                    Z          .00  SIDE SLOPE

                                                               ***
                         COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                 COMPUTATION TIME STEP
        ELEMENT     ALPHA       M          DT         DX        PEAK    TIME TO    VOLUME    MAXIMUM
                                                                          PEAK               CELERITY
                                          (MIN)      (FT)      (CFS)     (MIN)      (IN)      (FPS)

         MAIN       14.34      1.25         .32     105.00      2.56    780.16      1.01     11.05

                             INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL


         MAIN       14.34      1.25        5.00                 2.55    780.00      1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .7215E+00 EXCESS= .0000E+00 OUTFLOW= .7213E+00 BASIN STORAGE= .1864E-03 PERCENT ERROR=     .0


    ***          ***          ***          ***          ***

                    HYDROGRAPH AT STATION       CV4

  PEAK FLOW     TIME                    MAXIMUM AVERAGE FLOW
                                   6-HR        24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                        (CFS)
      3.       13.00                 1.          0.          0.           0.
                      (INCHES)     .681       1.009       1.009        1.009
                       (AC-FT)       0.          1.          1.           1.

                     CUMULATIVE AREA =      .01 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
 37 KK    *      HC4   *
          *            *
          **************

 39 KO       OUTPUT CONTROL VARIABLES
                IPRNT            3  PRINT CONTROL
                IPLOT            0  PLOT CONTROL
                QSCAL           0.  HYDROGRAPH PLOT SCALE
                IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                 IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
               TIMINT         .083  TIME INTERVAL IN HOURS


 38 HC       HYDROGRAPH COMBINATION
                ICOMP            2  NUMBER OF HYDROGRAPHS TO COMBINE
                                                               ***

    ***          ***          ***          ***          ***

                    HYDROGRAPH AT STATION       HC4

  PEAK FLOW     TIME                    MAXIMUM AVERAGE FLOW
                                   6-HR        24-HR       72 HR      23.92-HR
+   (CFS)       (HR)
                        (CFS)
+    23.       13.00                10.          4.          4.           4.
                      (INCHES)     .680       1.004       1.004        1.004
```

```
              (AC-FT)        5.          7.          7.          7.

              CUMULATIVE AREA =     .14 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


         **************
         *            *
 40 KK   *     SB46   *
         *            *
         **************

 44 KO         OUTPUT CONTROL VARIABLES
                     IPRNT          3  PRINT CONTROL
                     IPLOT          0  PLOT CONTROL
                     QSCAL         0.  HYDROGRAPH PLOT SCALE
                     IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                      IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                     ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                     ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                    TIMINT       .083  TIME INTERVAL IN HOURS


            SUBBASIN RUNOFF DATA

 41 BA         SUBBASIN CHARACTERISTICS
                     TAREA        .01  SUBBASIN AREA

               PRECIPITATION DATA

  8 PB               STORM       2.52  BASIN TOTAL PRECIPITATION

 10 PI            INCREMENTAL PRECIPITATION PATTERN
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .01     .01     .01     .01     .01     .01     .01     .01
                    .01     .01     .01     .01     .01     .01     .01     .01     .01     .01
                    .01     .01     .01     .01     .01     .01     .01     .01     .01     .01
                    .01     .01     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00

 42 LS         SCS LOSS RATE
                     STRTL        .44  INITIAL ABSTRACTION
                    CRVNBR      82.00  CURVE NUMBER
                     RTIMP        .00  PERCENT IMPERVIOUS AREA

 43 UD         SCS DIMENSIONLESS UNITGRAPH
                      TLAG        .08  LAG

                                                  ***

                                            UNIT HYDROGRAPH
                                         7 END-OF-PERIOD ORDINATES
             22.       22.        7         2.        1.        0.        0.

      ***           ***          ***          ***          ***

                  HYDROGRAPH AT STATION      SB46

   TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01
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```
PEAK FLOW     TIME                          MAXIMUM AVERAGE FLOW
                                 6-HR        24-HR        72-HR       23.92-HR
  (CFS)       (HR)
                       (CFS)
+      1.     13.00                 1.           0.           0.            0.
                       (INCHES)  .681        1.009        1.009         1.009
                       (AC-FT)     0.           0.           0.            0.

                       CUMULATIVE AREA =      .01 SQ MI
```

*** ***

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              **************
              *            *
  45 KK       *     CV32   *
              *            *
              **************

  47 KO          OUTPUT CONTROL VARIABLES
                        IPRNT          3  PRINT CONTROL
                        IPLOT          0  PLOT CONTROL
                        QSCAL         0.  HYDROGRAPH PLOT SCALE
                        IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                         IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                        ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                        ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                       TIMINT       .083  TIME INTERVAL IN HOURS

                HYDROGRAPH ROUTING DATA

  46 RD          MUSKINGUM-CUNGE CHANNEL ROUTING
                            L       255.  CHANNEL LENGTH
                            S      .2500  SLOPE
                            N       .030  CHANNEL ROUGHNESS COEFFICIENT
                           CA        .00  CONTRIBUTING AREA
                        SHAPE       CIRC  CHANNEL SHAPE
                           WD       1.50  BOTTOM WIDTH OR DIAMETER
                            Z        .00  SIDE SLOPE

                                                       ***
                                   COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                           COMPUTATION TIME STEP
                  ELEMENT    ALPHA       M         DT        DX        PEAK     TIME TO     VOLUME     MAXIMUM
                                                                                 PEAK                 CELERITY
                                                  (MIN)     (FT)      (CFS)     (MIN)        (IN)       (FPS)

                   MAIN      14.34     1.25        .44     127.47      1.33     779.97       1.01        9.71

                                           INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                   MAIN      14.34     1.25       5.00                 1.33     780.00       1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .3769E+00 EXCESS= .0000E+00 OUTFLOW= .3767E+00 BASIN STORAGE= .1346E-03 PERCENT ERROR=     .0


      ***              ***               ***              ***              ***

                       HYDROGRAPH AT STATION      CV32

PEAK FLOW     TIME                          MAXIMUM AVERAGE FLOW
                                 6-HR        24-HR        72-HR       23.92-HR
+  (CFS)      (HR)
                       (CFS)
+      1.     13.00                 1.           0.           0.            0.
                       (INCHES)  .681        1.009        1.009         1.009
                       (AC-FT)     0.           0.           0.            0.

                       CUMULATIVE AREA =      .01 SQ MI
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*** ***

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              **************
              *            *
  48 KK       *     SB47   *
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          *           *
          *************

 52 KO        OUTPUT CONTROL VARIABLES
                    IPRNT          3  PRINT CONTROL
                    IPLOT          0  PLOT CONTROL
                    QSCAL         0.  HYDROGRAPH PLOT SCALE
                    IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                     IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                    ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                    ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                   TIMINT       .083  TIME INTERVAL IN HOURS


             SUBBASIN RUNOFF DATA

 49 BA        SUBBASIN CHARACTERISTICS
                    TAREA        .01  SUBBASIN AREA

              PRECIPITATION DATA

  8 PB              STORM       2.52  BASIN TOTAL PRECIPITATION

 10 PI          INCREMENTAL PRECIPITATION PATTERN
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .01      .01      .01      .01      .01      .01      .01      .01
                      .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                      .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                      .01      .01      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                      .00      .00      .00      .00      .00      .00      .00

 50 LS        SCS LOSS RATE
                    STRTL        .44  INITIAL ABSTRACTION
                   CRVNBR      82.00  CURVE NUMBER
                    RTIMP        .00  PERCENT IMPERVIOUS AREA

 51 UD        SCS DIMENSIONLESS UNITGRAPH
                     TLAG        .08  LAG

                                                ***

                                          UNIT HYDROGRAPH
                                      7 END-OF-PERIOD ORDINATES
             19.      19.       6.       2.       1.       0.       0.

     ***          ***          ***          ***          ***

                  HYDROGRAPH AT STATION     SB47

    TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01

  PEAK FLOW      TIME                       MAXIMUM AVERAGE FLOW
                                  6-HR        24-HR       72-HR      23.92-HR
+   (CFS)        (HR)
                        (CFS)
+        1.     13.00                 0.          0.          0.          0.
                      (INCHES)      .681       1.009       1.009       1.009
                       (AC-FT)        0.          0.          0.          0.

                  CUMULATIVE AREA =      .01 SQ MI
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*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

           **************
           *            *
 53 KK     *     HC33   *
           *            *
           **************

 55 KO         OUTPUT CONTROL VARIABLES
                      IPRNT          3  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE
                      IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                       IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT       .083  TIME INTERVAL IN HOURS


 54 HC         HYDROGRAPH COMBINATION
                      ICOMP          2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                              ***

       ***            ***            ***            ***            ***

                         HYDROGRAPH AT STATION      HC33

 PEAK FLOW      TIME                         MAXIMUM AVERAGE FLOW
                                      6-HR        24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                           (CFS)
+       2.     13.00                    1.          0.          0.          0.
                        (INCHES)      .681       1.009       1.009       1.009
                         (AC-FT)        0.          1.          1.          1.

                        CUMULATIVE AREA =      .01 SQ MI



*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

           **************
           *            *
 56 KK     *      SB8   *
           *            *
           **************

 60 KO         OUTPUT CONTROL VARIABLES
                      IPRNT          3  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE
                      IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                       IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT       .083  TIME INTERVAL IN HOURS

              SUBBASIN RUNOFF DATA

 57 BA         SUBBASIN CHARACTERISTICS
                      TAREA        .25  SUBBASIN AREA

               PRECIPITATION DATA

  8 PB                STORM       2.52  BASIN TOTAL PRECIPITATION

 10 PI           INCREMENTAL PRECIPITATION PATTERN
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                   .00       .00       .01       .01       .01       .01       .01       .01       .01       .01
                   .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
```

.01	.01	.01	.01	.01	.01	.01	.01	.01	.01
.01	.01	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00			

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58 LS          SCS LOSS RATE
                  STRTL          .44  INITIAL ABSTRACTION
                 CRVNBR        82.00  CURVE NUMBER
                  RTIMP          .00  PERCENT IMPERVIOUS AREA

59 UD          SCS DIMENSIONLESS UNITGRAPH
                   TLAG          .27  LAG
```

***

UNIT HYDROGRAPH
18 END-OF-PERIOD ORDINATES

63.	212.	368.	393.	328.	220.	137.	91.	58.	38.
24.	16.	10.	7.	4.	3.	2.	1.		

***   ***   ***   ***   ***

HYDROGRAPH AT STATION SB8

TOTAL RAINFALL = 2.52, TOTAL LOSS = 1.51, TOTAL EXCESS = 1.01

	PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
				6-HR	24-HR	72-HR	23.92-HR
+	(CFS)	(HR)					
			(CFS)				
+	43.	13.08		19.	7.	7.	7.
			(INCHES)	.680	1.002	1.002	1.002
			(AC-FT)	9.	14.	14.	14.

CUMULATIVE AREA = .25 SQ MI

*** ***

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            **************
            *            *
61 KK       *     CV8    *
            *            *
            **************

63 KO          OUTPUT CONTROL VARIABLES
                  IPRNT            3  PRINT CONTROL
                  IPLOT            0  PLOT CONTROL
                  QSCAL           0.  HYDROGRAPH PLOT SCALE
                  IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                   IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                  ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                  ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                 TIMINT         .083  TIME INTERVAL IN HOURS

            HYDROGRAPH ROUTING DATA

62 RD          MUSKINGUM-CUNGE CHANNEL ROUTING
                      L         203.  CHANNEL LENGTH
                      S        .2500  SLOPE
                      N         .030  CHANNEL ROUGHNESS COEFFICIENT
                     CA          .00  CONTRIBUTING AREA
                  SHAPE         CIRC  CHANNEL SHAPE
                     WD         1.50  BOTTOM WIDTH OR DIAMETER
                      Z          .00  SIDE SLOPE
```

***

COMPUTED MUSKINGUM-CUNGE PARAMETERS
COMPUTATION TIME STEP

ELEMENT	ALPHA	M	DT	DX	PEAK	TIME TO PEAK	VOLUME	MAXIMUM CELERITY
			(MIN)	(FT)	(CFS)	(MIN)	(IN)	(FPS)

```
                 MAIN      14.34       1.25         .17     101.68      43.03     785.29       1.00       19.44

                                           INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                 MAIN      14.34       1.25        5.00                 42.97     785.00       1.00

CONTINUITY SUMMARY (AC-FT)    INFLOW= .1363E+02 EXCESS= .0000E+00 OUTFLOW= .1363E+02 BASIN STORAGE= .1903E-02 PERCENT ERROR=     .0


     ***            ***            ***            ***            ***

                      HYDROGRAPH AT STATION       CV8

 PEAK FLOW     TIME                          MAXIMUM AVERAGE FLOW
                                    6-HR        24-HR        72-HR      23.92-HR
+   (CFS)      (HR)
                          (CFS)
+     43.     13.08                  19.           7.           7.           7.
                        (INCHES)    .680        1.002        1.002        1.002
                         (AC-FT)      9.          14.          14.          14.

                        CUMULATIVE AREA =       .25 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

            **************
            *            *
  64 KK     *     SB24   *
            *            *
            **************

  68 KO          OUTPUT CONTROL VARIABLES
                      IPRNT          3  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE
                      IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                       IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT       .083  TIME INTERVAL IN HOURS


               SUBBASIN RUNOFF DATA

  65 BA          SUBBASIN CHARACTERISTICS
                      TAREA        .02  SUBBASIN AREA

                 PRECIPITATION DATA

   8 PB               STORM       2.52  BASIN TOTAL PRECIPITATION

  10 PI            INCREMENTAL PRECIPITATION PATTERN
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .01        .01        .01        .01        .01        .01        .01        .01
                       .01        .01        .01        .01        .01        .01        .01        .01        .01        .01
                       .01        .01        .01        .01        .01        .01        .01        .01        .01        .01
                       .01        .01        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                       .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
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                       .00        .00        .00        .00         00        .00        .00        .00        .00        .00
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            .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
            .00       .00       .00       .00       .00       .00       .00

 66 LS          SCS LOSS RATE
                  STRTL          .44  INITIAL ABSTRACTION
                 CRVNBR        52.00  CURVE NUMBER
                  RTIMP          .00  PERCENT IMPERVIOUS AREA

 67 UD          SCS DIMENSIONLESS UNITGRAPH
                   TLAG          .08  LAG

                                                          ***

                                                  UNIT HYDROGRAPH
                                               7 END-OF-PERIOD ORDINATES
          49.       49.       16.       5.        2.        1.        0.

     ***            ***          ***          ***          ***

                   HYDROGRAPH AT STATION     SB24

   TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01

  PEAK FLOW     TIME                          MAXIMUM AVERAGE FLOW
                                    6-HR       24-HR       72-HR     23.92-HR
+   (CFS)       (HR)
                         (CFS)
+      3.      13.00                  1.          0.          0.          0.
                       (INCHES)     .681       1.009       1.009       1.009
                        (AC-FT)       1.          1.          1.          1.

                  CUMULATIVE AREA =      .02 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

              **************
              *            *
  69 KK       *     HC8    *
              *            *
              **************

  71 KO         OUTPUT CONTROL VARIABLES
                    IPRNT          3  PRINT CONTROL
                    IPLOT          0  PLOT CONTROL
                    QSCAL         0.  HYDROGRAPH PLOT SCALE
                    IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                     IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                    ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                    ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                   TIMINT       .083  TIME INTERVAL IN HOURS


  70 HC         HYDROGRAPH COMBINATION
                    ICOMP          2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                          ***

     ***            ***          ***          ***          ***

                   HYDROGRAPH AT STATION      HC8

  PEAK FLOW     TIME                          MAXIMUM AVERAGE FLOW
                                    6-HR       24-HR       72-HR     23.92-HR
+   (CFS)       (HR)
                         (CFS)
+     45.      13.08                 20.          7.          7.          7.
                       (INCHES)     .680       1.003       1.003       1.003
                        (AC-FT)      10.         14.         14.         14.

                  CUMULATIVE AREA =      .27 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

              **************
              *            *
  72 KK       *     CV9    *
              *            *
              **************
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74 KO         OUTPUT CONTROL VARIABLES
                   IPRNT          3  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE
                   IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                   IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                   ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                   ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                  TIMINT       .083  TIME INTERVAL IN HOURS

             HYDROGRAPH ROUTING DATA

73 RD         MUSKINGUM-CUNGE CHANNEL ROUTING
                      L        206.  CHANNEL LENGTH
                      S       .2500  SLOPE
                      N        .030  CHANNEL ROUGHNESS COEFFICIENT
                     CA         .00  CONTRIBUTING AREA
                  SHAPE        CIRC  CHANNEL SHAPE
                     WD        1.50  BOTTOM WIDTH OR DIAMETER
                      Z         .00  SIDE SLOPE

                                                ***
                         COMPUTED MUSKINGUM-CUNGE PARAMETERS
                             COMPUTATION TIME STEP
    ELEMENT    ALPHA       M         DT        DX       PEAK     TIME TO     VOLUME     MAXIMUM
                                                                  PEAK                  CELERITY
                                   (MIN)      (FT)     (CFS)     (MIN)        (IN)       (FPS)

     MAIN      14.34      1.25        .17    103.13     45.42    785.13       1.00       19.65

                            INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

     MAIN      14.34      1.25       5.00               45.38    785.00       1.00

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1447E+02 EXCESS= .0000E+00 OUTFLOW= .1447E+02 BASIN STORAGE= .2025E-02 PERCENT ERROR=    .0


      ***          ***          ***          ***          ***

                  HYDROGRAPH AT STATION       CV9

  PEAK FLOW     TIME                       MAXIMUM AVERAGE FLOW
                                   6-HR       24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                        (CFS)
+      45.     13.08                20.          7.          7.          7.
                     (INCHES)      .680       1.003       1.003       1.003
                      (AC-FT)       10.         14.         14.         14.

                    CUMULATIVE AREA =     .27 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


            **************
            *            *
 75 KK      *     SB25   *
            *            *
            **************

 79 KO         OUTPUT CONTROL VARIABLES
                   IPRNT          3  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE
                   IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                   IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                   ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                   ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                  TIMINT       .083  TIME INTERVAL IN HOURS

             SUBBASIN RUNOFF DATA

 76 BA         SUBBASIN CHARACTERISTICS
                  TAREA         .01  SUBBASIN AREA

               PRECIPITATION DATA

  8 PB            STORM        2.52  BASIN TOTAL PRECIPITATION
```

```
10 PI            INCREMENTAL PRECIPITATION PATTERN
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .01     .01     .01     .01     .01     .01     .01     .01
                    .01     .01     .01     .01     .01     .01     .01     .01     .01     .01
                    .01     .01     .01     .01     .01     .01     .01     .01     .01     .01
                    .01     .01     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00

77 LS          SCS LOSS RATE
                  STRTL         .44  INITIAL ABSTRACTION
                 CRVNBR       82.00  CURVE NUMBER
                  RTIMP         .00  PERCENT IMPERVIOUS AREA

78 UD          SCS DIMENSIONLESS UNITGRAPH
                   TLAG         .08  LAG

                                                                 ***

                                                           UNIT HYDROGRAPH
                                                       7 END-OF-PERIOD ORDINATES
                 44.        45.        14.         5.         2.         1.         0.

         ***             ***           ***             ***              ***

                         HYDROGRAPH AT STATION     SB25

   TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01

  PEAK FLOW     TIME                             MAXIMUM AVERAGE FLOW
                                      6-HR        24-HR        72-HR       23.92-HR
+   (CFS)       (HR)
                          (CFS)
+        3.    13.00                     1.           0.           0.            0.
                        (INCHES)       .681        1.009        1.009         1.009
                         (AC-FT)         1.           1.           1.            1.

                        CUMULATIVE AREA =      .01 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

             **************
             *            *
80 KK        *      HC9   *
             *            *
             **************

82 KO          OUTPUT CONTROL VARIABLES
                  IPRNT           3  PRINT CONTROL
                  IPLOT           0  PLOT CONTROL
                  QSCAL          0.  HYDROGRAPH PLOT SCALE
                  IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                   IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                  ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                  ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                 TIMINT        .083  TIME INTERVAL IN HOURS


81 HC          HYDROGRAPH COMBINATION
                  ICOMP           2  NUMBER OF HYDROGRAPHS TO COMBINE
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                                                                                         ***

              ***                  ***                  ***                  ***                  ***

                                   HYDROGRAPH AT STATION       HC9

 PEAK FLOW     TIME                                   MAXIMUM AVERAGE FLOW
                                             6-HR         24-HR         72-HR        23.92-HR
+   (CFS)      (HR)
                                 (CFS)
+      48.     13.08                          21.            8.            8.              8.
                               (INCHES)      .680         1.003         1.003           1.003
                                (AC-FT)       10.           15.           15.             15.

                                CUMULATIVE AREA =       .28 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

              **************
              *            *
  83 KK       *     SB9    *
              *            *
              **************

  87 KO            OUTPUT CONTROL VARIABLES
                        IPRNT             3  PRINT CONTROL
                        IPLOT             0  PLOT CONTROL
                        QSCAL            0.  HYDROGRAPH PLOT SCALE
                        IPNCH             0  PUNCH COMPUTED HYDROGRAPH
                         IOUT            22  SAVE HYDROGRAPH ON THIS UNIT
                        ISAV1             1  FIRST ORDINATE PUNCHED OR SAVED
                        ISAV2           288  LAST ORDINATE PUNCHED OR SAVED
                       TIMINT          .083  TIME INTERVAL IN HOURS


                 SUBBASIN RUNOFF DATA

  84 BA            SUBBASIN CHARACTERISTICS
                        TAREA           .18  SUBBASIN AREA

                   PRECIPITATION DATA

   8 PB                 STORM          2.52  BASIN TOTAL PRECIPITATION

  10 PI              INCREMENTAL PRECIPITATION PATTERN
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .01        .01        .01        .01        .01        .01        .01        .01
                        .01        .01        .01        .01        .01        .01        .01        .01        .01        .01
                        .01        .01        .01        .01        .01        .01        .01        .01        .01        .01
                        .01        .01        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                        .00        .00        .00        .00        .00        .00        .00

  85 LS            SCS LOSS RATE
                        STRTL           .44  INITIAL ABSTRACTION
                       CRVNBR         82.00  CURVE NUMBER
                        RTIMP           .00  PERCENT IMPERVIOUS AREA

  86 UD            SCS DIMENSIONLESS UNITGRAPH
                         TLAG           .27  LAG
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                                                   ***

                                          UNIT HYDROGRAPH
                                       18 END-OF-PERIOD ORDINATES
          46.      152.      264.      282.      235.      158.       98.       65.       41.       27.
          17.       11.        7.        5.        3.        2.        1.        1.

   ***          ***           ***           ***           ***

                        HYDROGRAPH AT STATION     SB9

  TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01

 PEAK FLOW     TIME                            MAXIMUM AVERAGE FLOW
                                   6-HR        24-HR       72-HR      23.92-HR
+   (CFS)      (HR)
                          (CFS)
+    31.      13.08                  13.          5.          5.          5.
                        (INCHES)    .680       1.002       1.002       1.002
                        (AC-FT)       7.         10.         10.         10.

                       CUMULATIVE AREA =     .18 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

          **************
          *            *
 88 KK    *      CV13  *
          *            *
          **************

 90 KO          OUTPUT CONTROL VARIABLES
                      IPRNT           3  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                       IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT        .083  TIME INTERVAL IN HOURS

              HYDROGRAPH ROUTING DATA

 89 RD          MUSKINGUM-CUNGE CHANNEL ROUTING
                          L          207.  CHANNEL LENGTH
                          S         .2500  SLOPE
                          N          .030  CHANNEL ROUGHNESS COEFFICIENT
                         CA           .00  CONTRIBUTING AREA
                      SHAPE          CIRC  CHANNEL SHAPE
                         WD          1.50  BOTTOM WIDTH OR DIAMETER
                          Z           .00  SIDE SLOPE

                                                   ***
                               COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                    COMPUTATION TIME STEP
                 ELEMENT    ALPHA       M          DT         DX        PEAK      TIME TO      VOLUME     MAXIMUM
                                                                                   PEAK                   CELERITY
                                                  (MIN)      (FT)       (CFS)      (MIN)       (IN)        (FPS)

                   MAIN      14.34      1.25        .19     103.29      30.86     785.29       1.00       18.19

                                    INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                   MAIN      14.34      1.25       5.00                 30.83     785.00       1.00

CONTINUITY SUMMARY (AC-FT) - INFLOW= .9773E+01 EXCESS= .0000E+00 OUTFLOW= .9772E+01 BASIN STORAGE= .1482E-02 PERCENT ERROR=     .0


   ***          ***           ***           ***           ***

                        HYDROGRAPH AT STATION     CV13

 PEAK FLOW     TIME                            MAXIMUM AVERAGE FLOW
                                   6-HR        24-HR       72-HR      23.92-HR
+   (CFS)      (HR)
                          (CFS)
     31.      13.08                  13.          5.          5.          5.
                        (INCHES)    .680       1.002       1.002       1.002
```

```
                 (AC-FT)          7.          10.          10.          10.

                 CUMULATIVE AREA =     .18 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

          **************
          *            *
 91 KK    *    SB29    *
          *            *
          **************

 95 KO          OUTPUT CONTROL VARIABLES
                      IPRNT          3  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE
                      IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                       IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT       .083  TIME INTERVAL IN HOURS


              SUBBASIN RUNOFF DATA

 92 BA          SUBBASIN CHARACTERISTICS
                      TAREA        .02  SUBBASIN AREA

                PRECIPITATION DATA

  8 PB                STORM       2.52  BASIN TOTAL PRECIPITATION

 10 PI            INCREMENTAL PRECIPITATION PATTERN
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .01     .01     .01     .01     .01     .01     .01     .01
                      .01     .01     .01     .01     .01     .01     .01     .01     .01     .01
                      .01     .01     .01     .01     .01     .01     .01     .01     .01     .01
                      .01     .01     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                      .00     .00     .00     .00     .00     .00     .00

 93 LS          SCS LOSS RATE
                      STRTL        .44  INITIAL ABSTRACTION
                     CRVNBR      82.00  CURVE NUMBER
                      RTIMP        .00  PERCENT IMPERVIOUS AREA

 94 UD          SCS DIMENSIONLESS UNITGRAPH
                       TLAG        .08  LAG

                                                  ***

                                            UNIT HYDROGRAPH
                                         7 END-OF-PERIOD ORDINATES
             49.       49.       16.        5.        2.        1.        0.

     ***          ***          ***          ***          ***

                 HYDROGRAPH AT STATION      SB29

  TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01
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 PEAK FLOW      TIME                               MAXIMUM AVERAGE FLOW
                                        6-HR         24-HR        72-HR       23.92-HR
   (CFS)        (HR)
                             (CFS)
+        3.     13.00                      1.            0.           0.            0.
                            (INCHES)     .681         1.009        1.009         1.009
                             (AC-FT)        1.            1.           1.            1.

                             CUMULATIVE AREA =      .02 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

              **************
              *            *
   96 KK      *     HC13   *
              *            *
              **************

   98 KO           OUTPUT CONTROL VARIABLES
                         IPRNT            3  PRINT CONTROL
                         IPLOT            0  PLOT CONTROL
                         QSCAL           0.  HYDROGRAPH PLOT SCALE
                         IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                          IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                         ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                         ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                        TIMINT         .083  TIME INTERVAL IN HOURS


   97 HC           HYDROGRAPH COMBINATION
                         ICOMP            2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                                        ***

      ***              ***            ***              ***              ***

                              HYDROGRAPH AT STATION     HC13

 PEAK FLOW      TIME                               MAXIMUM AVERAGE FLOW
                                        6-HR         24-HR        72-HR       23.92-HR
   (CFS)        (HR)
                             (CFS)
+       33.     13.08                     15.            5.           5.            5.
                            (INCHES)     .680         1.003        1.003         1.003
                             (AC-FT)        7.           11.          11.           11.

                             CUMULATIVE AREA =      .20 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

              **************
              *            *
   99 KK      *     CV14   *
              *            *
              **************

  101 KO           OUTPUT CONTROL VARIABLES
                         IPRNT            3  PRINT CONTROL
                         IPLOT            0  PLOT CONTROL
                         QSCAL           0.  HYDROGRAPH PLOT SCALE
                         IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                          IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                         ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                         ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                        TIMINT         .083  TIME INTERVAL IN HOURS

                 HYDROGRAPH ROUTING DATA

  100 RD           MUSKINGUM-CUNGE CHANNEL ROUTING
                             L         202.  CHANNEL LENGTH
                             S        .2500  SLOPE
                             N         .030  CHANNEL ROUGHNESS COEFFICIENT
                            CA          .00  CONTRIBUTING AREA
                         SHAPE         CIRC  CHANNEL SHAPE
                            WD         1.50  BOTTOM WIDTH OR DIAMETER
                             Z          .00  SIDE SLOPE

                                                                        ***
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```
                              COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                     COMPUTATION TIME STEP
                ELEMENT   ALPHA     M        DT        DX       PEAK    TIME TO    VOLUME    MAXIMUM
                                                                          PEAK                CELERITY
                                           (MIN)      (FT)     (CFS)     (MIN)      (IN)      (FPS)

                 MAIN     14.34    1.25      .19     100.87    33.28    785.20      1.00      18.46

                                   INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                 MAIN     14.34    1.25     5.00               33.26    785.00      1.00

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1062E-02 EXCESS= .0000E+00 OUTFLOW= .1062E+02 BASIN STORAGE= .1546E-02 PERCENT ERROR=    .0


     ***           ***           ***           ***           ***

                 HYDROGRAPH AT STATION      CV14

  PEAK FLOW      TIME                       MAXIMUM AVERAGE FLOW
                                  6-HR         24-HR        72-HR        23.92-HR
+    (CFS)       (HR)
                       (CFS)
+      33.      13.08              15.           5.           5.            5.
                     (INCHES)     .680        1.003        1.003         1.003
                      (AC-FT)       7.          11.          11.           11.

                 CUMULATIVE AREA =       .20 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

           **************
           *            *
  102 KK   *     SB30   *
           *            *
           **************

  106 KO        OUTPUT CONTROL VARIABLES
                     IPRNT          3  PRINT CONTROL
                     IPLOT          0  PLOT CONTROL
                     QSCAL         0.  HYDROGRAPH PLOT SCALE
                     IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                      IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                     ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                     ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                    TIMINT       .083  TIME INTERVAL IN HOURS

              SUBBASIN RUNOFF DATA

  103 BA        SUBBASIN CHARACTERISTICS
                     TAREA        .01  SUBBASIN AREA

                PRECIPITATION DATA

    8 PB             STORM       2.52  BASIN TOTAL PRECIPITATION

   10 PI          INCREMENTAL PRECIPITATION PATTERN
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .01        .01        .01        .01        .01        .01        .01        .01
                      .01        .01        .01        .01        .01        .01        .01        .01        .01        .01
                      .01        .01        .01        .01        .01        .01        .01        .01        .01        .01
                      .01        .01        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
                      .00        .00        .00        .00        .00        .00        .00        .00        .00        .00
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               .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
               .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
               .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
               .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
               .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
               .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
               .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
               .00      .00      .00      .00      .00      .00      .00

 104 LS       SCS LOSS RATE
                   STRTL         .44  INITIAL ABSTRACTION
                  CRVNBR       82.00  CURVE NUMBER
                   RTIMP         .00  PERCENT IMPERVIOUS AREA

 105 UD       SCS DIMENSIONLESS UNITGRAPH
                    TLAG         .08  LAG

                                                        ***

                                                     UNIT HYDROGRAPH
                                                 7 END-OF-PERIOD ORDINATES
            44.       45.       14.        5.        2.        1.        0.

      ***            ***            ***            ***            ***

                        HYDROGRAPH AT STATION     SB30

    TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01

  PEAK FLOW     TIME                        MAXIMUM AVERAGE FLOW
                                  6-HR       24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                          (CFS)
+       3.     13.00                 1.          0.          0.           0.
                        (INCHES)   .681       1.009       1.009        1.009
                        (AC-FT)      1.          1.          1.           1.

                       CUMULATIVE AREA =      .01 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


             **************
             *            *
 107 KK      *    HC14    *
             *            *
             **************

 109 KO       OUTPUT CONTROL VARIABLES
                   IPRNT           3  PRINT CONTROL
                   IPLOT           0  PLOT CONTROL
                   QSCAL          0.  HYDROGRAPH PLOT SCALE
                   IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                    IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                   ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                   ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                  TIMINT        .083  TIME INTERVAL IN HOURS

 108 HC       HYDROGRAPH COMBINATION
                   ICOMP           2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                        ***

      ***            ***            ***            ***            ***

                        HYDROGRAPH AT STATION     HC14

  PEAK FLOW     TIME                        MAXIMUM AVERAGE FLOW
                                  6-HR       24-HR       72-HR      23.92-HR
+   (CFS)       (HR)
                          (CFS)
+      36.     13.08                16.          6.          6.           6.
                        (INCHES)   .680       1.003       1.003        1.003
                        (AC-FT)      8.         11.         11.          11.

                       CUMULATIVE AREA =      .21 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***
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            **************
            *            *
 110 KK     *     SB10   *
            *            *
            **************

114 KO           OUTPUT CONTROL VARIABLES
                       IPRNT          3  PRINT CONTROL
                       IPLOT          0  PLOT CONTROL
                       QSCAL         0.  HYDROGRAPH PLOT SCALE
                       IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                        IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                       ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                       ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                      TIMINT       .083  TIME INTERVAL IN HOURS


               SUBBASIN RUNOFF DATA

111 BA           SUBBASIN CHARACTERISTICS
                       TAREA        .13  SUBBASIN AREA

                 PRECIPITATION DATA

  8 PB                 STORM       2.52  BASIN TOTAL PRECIPITATION

 10 PI             INCREMENTAL PRECIPITATION PATTERN
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .01       .01       .01       .01       .01       .01       .01       .01
                       .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                       .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                       .01       .01       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                       .00       .00       .00       .00       .00       .00       .00

112 LS           SCS LOSS RATE
                       STRTL        .44  INITIAL ABSTRACTION
                      CRVNBR      82.00  CURVE NUMBER
                       RTIMP        .00  PERCENT IMPERVIOUS AREA

113 UD           SCS DIMENSIONLESS UNITGRAPH
                        TLAG        .19  LAG

                                                         ***

                                                    UNIT HYDROGRAPH
                                                13 END-OF-PERIOD ORDINATES
                 73.       235.      278.      207.      109.       62.       35.       19.       11.        6.
                  3.         2.        1.

       ***            ***            ***            ***            ***

                          HYDROGRAPH AT STATION      SB10

   TOTAL RAINFALL =     2.52, TOTAL LOSS =     1.51, TOTAL EXCESS =     1.01

 PEAK FLOW      TIME                       MAXIMUM AVERAGE FLOW
                                  6-HR         24 HR        72-HR        23.92-HR
+  (CFS)        (HR)
                          (CFS)
+      24.      13.08                10.           4           4.           4.
                        (INCHES)    .681       1.005        1.005        1.005
                        (AC-FT)       5.           7.           7.           7.

                        CUMULATIVE AREA =      .13 SQ MI
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*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

          **************
          *            *
 115 KK   *     CV18   *
          *            *
          **************

 117 KO        OUTPUT CONTROL VARIABLES
                     IPRNT          3  PRINT CONTROL
                     IPLOT          0  PLOT CONTROL
                     QSCAL         0.  HYDROGRAPH PLOT SCALE
                     IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                      IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                     ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                     ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                    TIMINT       .083  TIME INTERVAL IN HOURS

             HYDROGRAPH ROUTING DATA

 116 RD        MUSKINGUM-CUNGE CHANNEL ROUTING
                         L       205.  CHANNEL LENGTH
                         S      .2500  SLOPE
                         N       .030  CHANNEL ROUGHNESS COEFFICIENT
                        CA        .00  CONTRIBUTING AREA
                     SHAPE       CIRC  CHANNEL SHAPE
                        WD       1.50  BOTTOM WIDTH OR DIAMETER
                         Z        .00  SIDE SLOPE

                                                        ***
                           COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                  COMPUTATION TIME STEP
                 ELEMENT    ALPHA      M        DT        DX      PEAK    TIME TO     VOLUME    MAXIMUM
                                                                           PEAK                 CELERITY
                                               (MIN)     (FT)     (CFS)    (MIN)       (IN)      (FPS)

                   MAIN     14.34     1.25       .20    102.48    23.88    784.95      1.01      17.28

                                  INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                   MAIN     14.34     1.25      5.00              23.88    785.00      1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .7212E+01 EXCESS= .0000E+00 OUTFLOW= .7211E+01 BASIN STORAGE= .1151E-02 PERCENT ERROR=     .0


      ***          ***          ***          ***          ***

                      HYDROGRAPH AT STATION    CV18

 PEAK FLOW     TIME                         MAXIMUM AVERAGE FLOW
                                  6-HR       24-HR       72-HR       23.92-HR
+    (CFS)     (HR)
                       (CFS)
+     24.     13.08                10.          4.          4.           4.
                     (INCHES)     .680       1.005       1.005        1.005
                      (AC-FT)       5.          7.          7.           7.

                     CUMULATIVE AREA =     .13 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

          **************
          *            *
 118 KK   *     SB34   *
          *            *
          **************

 122 KO        OUTPUT CONTROL VARIABLES
                     IPRNT          3  PRINT CONTROL
                     IPLOT          0  PLOT CONTROL
                     QSCAL         0.  HYDROGRAPH PLOT SCALE
                     IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                      IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                     ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
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                    ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                   TIMINT         .083  TIME INTERVAL IN HOURS


            SUBBASIN RUNOFF DATA

 119 BA       SUBBASIN CHARACTERISTICS
                    TAREA          .02  SUBBASIN AREA

              PRECIPITATION DATA

   8 PB             STORM         2.52  BASIN TOTAL PRECIPITATION

  10 PI         INCREMENTAL PRECIPITATION PATTERN
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .01     .01     .01     .01     .01     .01     .01     .01
                    .01     .01     .01     .01     .01     .01     .01     .01     .01     .01
                    .01     .01     .01     .01     .01     .01     .01     .01     .01     .01
                    .01     .01     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                    .00     .00     .00     .00     .00     .00     .00

  20 LS       SCS LOSS RATE
                   STRTL           .44  INITIAL ABSTRACTION
                  CRVNBR         82.00  CURVE NUMBER
                   RTIMP           .00  PERCENT IMPERVIOUS AREA

 121 UD       SCS DIMENSIONLESS UNITGRAPH
                    TLAG           .08  LAG

                                                  ***

                                               UNIT HYDROGRAPH
                                            7 END-OF-PERIOD ORDINATES
            49.      50.      16.       5.       2.       1.       0.

    ***          ***          ***          ***          ***

                  HYDROGRAPH AT STATION     SB34

  TOTAL RAINFALL =   2.52, TOTAL LOSS =   1.51, TOTAL EXCESS =   1.01

 PEAK FLOW     TIME                      MAXIMUM AVERAGE FLOW
                                 6-HR        24-HR       72-HR      23.92-HR
+  (CFS)       (HR)
                      (CFS)
+       3.    13.00                1.          0.          0.          0.
                    (INCHES)     .681       1.009       1.009       1.009
                     (AC-FT)       1.          1.          1.          1.

                    CUMULATIVE AREA =     .02 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


            **************
            *            *
 123 KK     *     HC18   *
            *            *
            **************

  25 KO       OUTPUT CONTROL VARIABLES
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```
                   IPRNT          3  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE
                   IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                    IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                   ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                   ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                  TIMINT       .083  TIME INTERVAL IN HOURS

  124 HC        HYDROGRAPH COMBINATION
                   ICOMP          2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                ***

     ***        ***        ***        ***        ***

                    HYDROGRAPH AT STATION     HC18

  PEAK FLOW    TIME                      MAXIMUM AVERAGE FLOW
                                  6-HR        24-HR      72-HR      23.92-HR
+  (CFS)       (HR)
                       (CFS)
+     27.     13.00                11.          4.         4.           4.
                     (INCHES)     .680       1.006      1.006        1.006
                      (AC-FT)       5.          8.         8.           8.

                    CUMULATIVE AREA =     .15 SQ MI

*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

             **************
             *            *
  126 KK     *     CV19   *
             *            *
             **************

  128 KO        OUTPUT CONTROL VARIABLES
                   IPRNT          3  PRINT CONTROL
                   IPLOT          0  PLOT CONTROL
                   QSCAL         0.  HYDROGRAPH PLOT SCALE
                   IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                    IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                   ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                   ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                  TIMINT       .083  TIME INTERVAL IN HOURS

             HYDROGRAPH ROUTING DATA

  127 RD        MUSKINGUM-CUNGE CHANNEL ROUTING
                       L       207.  CHANNEL LENGTH
                       S      .2500  SLOPE
                       N       .030  CHANNEL ROUGHNESS COEFFICIENT
                      CA        .00  CONTRIBUTING AREA
                   SHAPE       CIRC  CHANNEL SHAPE
                      WD       1.50  BOTTOM WIDTH OR DIAMETER
                       Z        .00  SIDE SLOPE

                                                ***
                            COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                   COMPUTATION TIME STEP
                 ELEMENT    ALPHA      M         DT       DX      PEAK    TIME TO    VOLUME    MAXIMUM
                                                                            PEAK                CELERITY
                                               (MIN)     (FT)    (CFS)    (MIN)      (IN)      (FPS)

                   MAIN     14.34     1.25        .19   103.29    26.77    780.25      1.01     17.68

                                  INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                   MAIN     14.34     1.25       5.00             26.73    780.00      1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .8061E+01 EXCESS= .0000E+00 OUTFLOW= .8060E+01 BASIN STORAGE= .1268E-02 PERCENT ERROR=    .0

     ***        ***        ***        ***        ***

                    HYDROGRAPH AT STATION     CV19
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 EAK FLOW     TIME                          MAXIMUM AVERAGE FLOW
                                     6-HR        24-HR       72-HR      23.92-HR
  (CFS)       (HR.
                          (CFS)
+     27.     13.00                   11.           4.          4.            4.
                       (INCHES)      .680       1.005       1.005         1.005
                        (AC-FT)        5.          8.          8.            8.

                       CUMULATIVE AREA =      .15 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


             **************
             *            *
 129 KK      *     SB35   *
             *            *
             **************

 133 KO          OUTPUT CONTROL VARIABLES
                      IPRNT           3  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                       IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT        .083  TIME INTERVAL IN HOURS


               SUBBASIN RUNOFF DATA

 130 BA          SUBBASIN CHARACTERISTICS
                      TAREA         .01  SUBBASIN AREA

                 PRECIPITATION DATA

   8 PB               STORM        2.52  BASIN TOTAL PRECIPITATION

  10 PI            INCREMENTAL PRECIPITATION PATTERN
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .01      .01      .01      .01      .01      .01      .01      .01
                       .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                       .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                       .01      .01      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                       .00      .00      .00      .00      .00      .00      .00

 131 LS          SCS LOSS RATE
                      STRTL         .44  INITIAL ABSTRACTION
                     CRVNBR       82.00  CURVE NUMBER
                      RTIMP         .00  PERCENT IMPERVIOUS AREA

 132 UD          SCS DIMENSIONLESS UNITGRAPH
                       TLAG         .08  LAG

                                                         ***

                                                    UNIT HYDROGRAPH
                                                7 END-OF-PERIOD ORDINATES
               44.       45.       14.        5.        2.        1.        0
```

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     ***              ***              ***              ***              ***

                         HYDROGRAPH AT STATION      SB35

   TOTAL RAINFALL =    2.52, TOTAL LOSS =     1.51, TOTAL EXCESS =     1.01

  PEAK FLOW      TIME                            MAXIMUM AVERAGE FLOW
                                        6-HR        24-HR       72-HR      23.92-HR
+   (CFS)        (HR)
                             (CFS)
+        3.     13.00                     1.          0.          0.           0.
                           (INCHES)     .681       1.009       1.009        1.009
                            (AC-FT)       1.          1.          1.           1.

                         CUMULATIVE AREA =      .01 SQ MI
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*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

              **************
              *            *
  134 KK      *     HC19   *
              *            *
              **************

  136 KO          OUTPUT CONTROL VARIABLES
                       IPRNT            3  PRINT CONTROL
                       IPLOT            0  PLOT CONTROL
                       QSCAL           0.  HYDROGRAPH PLOT SCALE
                       IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                        IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                       ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                       ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                      TIMINT         .083  TIME INTERVAL IN HOURS


  135 HC          HYDROGRAPH COMBINATION
                       ICOMP            2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                                    ***

     ***              ***              ***              ***              ***

                         HYDROGRAPH AT STATION      HC19

  PEAK FLOW      TIME                            MAXIMUM AVERAGE FLOW
                                        6-HR        24-HR       72-HR      23.92-HR
+   (CFS)        (HR)
                             (CFS)
+       29.     13.00                    12.          4.          4.           4.
                           (INCHES)     .680       1.006       1.006        1.006
                            (AC-FT)       6.          9.          9.           9.

                         CUMULATIVE AREA =      .16 SQ MI
```

```
*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

              **************
              *            *
  137 KK      *     SB11   *
              *            *
              **************

  141 KO          OUTPUT CONTROL VARIABLES
                       IPRNT            3  PRINT CONTROL
                       IPLOT            0  PLOT CONTROL
                       QSCAL           0.  HYDROGRAPH PLOT SCALE
                       IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                        IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                       ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                       ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                      TIMINT         .083  TIME INTERVAL IN HOURS

                 SUBBASIN RUNOFF DATA

  138 BA          SUBBASIN CHARACTERISTICS
                       TAREA          .19  SUBBASIN AREA

                  PRECIPITATION DATA
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```
  8 PB            STORM       2.52  BASIN TOTAL PRECIPITATION

 10 PI         INCREMENTAL PRECIPITATION PATTERN
                  .00    .00    .00    .00    .00    .00    .00    .00    .00    .00
                  .00    .00    .00    .00    .00    .00    .00    .00    .00    .00
                  .00    .00    .00    .00    .00    .00    .00    .00    .00    .00
                  .00    .00    .00    .00    .00    .00    .00    .00    .00    .00
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                  .00    .00    .01    .01    .01    .01    .01    .01    .01    .01
                  .01    .01    .01    .01    .01    .01    .01    .01    .01    .01
                  .01    .01    .01    .01    .01    .01    .01    .01    .01    .01
                  .01    .01    .00    .00    .00    .00    .00    .00    .00    .00
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                  .00    .00    .00    .00    .00    .00    .00    .00    .00    .00
                  .00    .00    .00    .00    .00    .00    .00

139 LS         SCS LOSS RATE
                 STRTL         .44  INITIAL ABSTRACTION
                 CRVNBR      82.00  CURVE NUMBER
                 RTIMP         .00  PERCENT IMPERVIOUS AREA

140 UD         SCS DIMENSIONLESS UNITGRAPH
                 TLAG          .19  LAG

                                            ***

                                        UNIT HYDROGRAPH
                                    13 END-OF-PERIOD ORDINATES
            102.     328.     387.     289.     153.      87.      48.      27.      15.       8.
              5.       3.       1.

       ***          ***          ***          ***          ***

                    HYDROGRAPH AT STATION      SB11

   TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01

  PEAK FLOW      TIME                        MAXIMUM AVERAGE FLOW
                                     6-HR         24-HR        72-HR      23.92-HR
+   (CFS)       (HR)
                        (CFS)
+      33.      13.08                 14.           5.           5.          5.
                       (INCHES)      .681        1.005        1.005       1.005
                        (AC-FT)        7.          10.          10.         10.

                    CUMULATIVE AREA =      .19 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


          **************
          *            *
142 KK    *     CV23   *
          *            *
          **************

144 KO         OUTPUT CONTROL VARIABLES
                 IPRNT           3  PRINT CONTROL
                 IPLOT           0  PLOT CONTROL
                 QSCAL          0.  HYDROGRAPH PLOT SCALE
                 IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                 IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                 ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                 ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                 TIMINT       .083  TIME INTERVAL IN HOURS
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            HYDROGRAPH ROUTING DATA

143 RD        MUSKINGUM-CUNGE CHANNEL ROUTING
                   L         207.  CHANNEL LENGTH
                   S        .2500  SLOPE
                   N         .030  CHANNEL ROUGHNESS COEFFICIENT
                  CA          .00  CONTRIBUTING AREA
               SHAPE         CIRC  CHANNEL SHAPE
                  WD         1.50  BOTTOM WIDTH OR DIAMETER
                   Z          .00  SIDE SLOPE

                                              ***
                      COMPUTED MUSKINGUM-CUNGE PARAMETERS
                          COMPUTATION TIME STEP
      ELEMENT     ALPHA      M        DT        DX      PEAK    TIME TO    VOLUME    MAXIMUM
                                                                  PEAK               CELERITY
                                     (MIN)     (FT)     (CFS)    (MIN)      (IN)      (FPS)

       MAIN       14.34     1.25       .19    103.29    33.29    785.00     1.01      18.46

                           INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

       MAIN       14.34     1.25      5.00              33.29    785.00     1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1005E+02 EXCESS= .0000E+00 OUTFLOW= .1005E+02 BASIN STORAGE= .1513E-02 PERCENT ERROR=     .0


     ***          ***          ***          ***          ***

                  HYDROGRAPH AT STATION     CV23

  PEAK FLOW     TIME                    MAXIMUM AVERAGE FLOW
                                   6-HR       24-HR      72-HR     23.92-HR
+   (CFS)       (HR)
                          (CFS)
+      33.     13.08                14.          5.         5.          5.
                        (INCHES)   .681       1.005      1.005       1.005
                         (AC-FT)     7.         10.        10.         10.

                      CUMULATIVE AREA =      .19 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


         **************
         *            *
145 KK   *     SB39   *
         *            *
         **************

149 KO        OUTPUT CONTROL VARIABLES
                  IPRNT          3  PRINT CONTROL
                  IPLOT          0  PLOT CONTROL
                  QSCAL         0.  HYDROGRAPH PLOT SCALE
                  IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                   IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                  ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                  ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                 TIMINT       .083  TIME INTERVAL IN HOURS


            SUBBASIN RUNOFF DATA

146 BA        SUBBASIN CHARACTERISTICS
                  TAREA        .02  SUBBASIN AREA

              PRECIPITATION DATA

  8 PB            STORM       2.52  BASIN TOTAL PRECIPITATION

 10 PI          INCREMENTAL PRECIPITATION PATTERN
                   .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                   .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                   .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                   .00     .00     .00     .00     .00     .00      00     .00     .00     .00
                   .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                   .00     .00     .00     .00     .00     .00      00     .00     .00     .00
                   .00     .00     .00     .00     .00     .00      00     .00     .00     .00
                   .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
                   .00     .00     .00     .00     .00     .00      00     .00     .00     .00
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.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.01	.01	.01	.01	.01	.01	.01	.01
.01	.01	.01	.01	.01	.01	.01	.01	.01	.01
.01	.01	.01	.01	.01	.01	.01	.01	.01	.01
.01	.01	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
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.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
.00	.00	.00	.00	.00	.00	.00			

```
147 LS          SCS LOSS RATE
                    STRTL          .44  INITIAL ABSTRACTION
                   CRVNBR        82.00  CURVE NUMBER
                    RTIMP          .00  PERCENT IMPERVIOUS AREA

148 UD          SCS DIMENSIONLESS UNITGRAPH
                     TLAG          .08  LAG

                                                                ***

                                                         UNIT HYDROGRAPH
                                                     7 END-OF-PERIOD ORDINATES
                 48        49.        16.        5.         2.         1.         0.

     ***             ***             ***             ***             ***

                         HYDROGRAPH AT STATION      SB39

   TOTAL RAINFALL =     2.52, TOTAL LOSS =     1.51, TOTAL EXCESS =     1.01

PEAK FLOW      TIME                            MAXIMUM AVERAGE FLOW
                                       6-HR          24-HR         72-HR       23.92-HR

  (CFS)        (HR)
                          (CFS)
      3.      13.00                      1.             0.            0.             0.
                       (INCHES)        .681          1.009         1.009          1.009
                        (AC-FT)          1.             1.            1.             1.

                        CUMULATIVE AREA =       .02 SQ MI
```

*** ***

```
            **************
            *            *
150 KK      *      HC23  *
            *            *
            **************

152 KO          OUTPUT CONTROL VARIABLES
                    IPRNT            3  PRINT CONTROL
                    IPLOT            0  PLOT CONTROL
                    QSCAL           0.  HYDROGRAPH PLOT SCALE
                    IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                     IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                    ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                    ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                   TIMINT         .083  TIME INTERVAL IN HOURS


151 HC          HYDROGRAPH COMBINATION
                    ICOMP            2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                                ***

     ***             ***             ***             ***             ***

                         HYDROGRAPH AT STATION      HC23

PEAK FLOW      TIME                            MAXIMUM AVERAGE FLOW
                                       6-HR          24-HR         72-HR       23.92 HR

  (CFS)         HR
                          CFS)
```

```
     36.     13.00                  15.          6.          6.          6.
                       (INCHES)    .681       1.006       1.006       1.006
                        (AC-FT)      7.         11.         11.         11.

                        CUMULATIVE AREA =     .20 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


            **************
            *            *
  153 KK    *     CV24   *
            *            *
            **************

  155 KO         OUTPUT CONTROL VARIABLES
                      IPRNT           3  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                       IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT        .083  TIME INTERVAL IN HOURS

             HYDROGRAPH ROUTING DATA

  154 RD         MUSKINGUM-CUNGE CHANNEL ROUTING
                          L        202.  CHANNEL LENGTH
                          S       .2500  SLOPE
                          N        .030  CHANNEL ROUGHNESS COEFFICIENT
                         CA         .00  CONTRIBUTING AREA
                      SHAPE        CIRC  CHANNEL SHAPE
                         WD        1.50  BOTTOM WIDTH OR DIAMETER
                          Z         .00  SIDE SLOPE

                                                                   ***
                                      COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                             COMPUTATION TIME STEP
                 ELEMENT    ALPHA        M          DT         DX        PEAK     TIME TO     VOLUME    MAXIMUM
                                                                                    PEAK                CELERITY
                                                   (MIN)      (FT)      (CFS)      (MIN)       (IN)      (FPS)

                   MAIN     14.34      1.25         .18     101.04     36.09     780.25       1.01      18.77

                                                INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL

                   MAIN     14.34      1.25        5.00                36.04     780.00       1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1089E+02 EXCESS= .0000E+00 OUTFLOW= .1089E+02 BASIN STORAGE= .1578E-02 PERCENT ERROR=    .0


      ***            ***           ***           ***           ***

                         HYDROGRAPH AT STATION     CV24

  PEAK FLOW     TIME                          MAXIMUM AVERAGE FLOW
                                    6-HR        24-HR       72-HR      23.92-HR
 +   (CFS)      (HR)
                            (CFS)
 +     36.     13.00                  15.          6.          6.          6.
                       (INCHES)    .680       1.005       1.005       1.005
                        (AC-FT)      7.         11.         11.         11.

                        CUMULATIVE AREA =     .20 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


            **************
            *            *
  156 KK    *     SB40   *
            *            *
            **************

   50 KO         OUTPUT CONTROL VARIABLES
```

```
                    IPRNT            3  PRINT CONTROL
                    IPLOT            0  PLOT CONTROL
                    QSCAL           0.  HYDROGRAPH PLOT SCALE
                    IPNCH            0  PUNCH COMPUTED HYDROGRAPH
                     IOUT           22  SAVE HYDROGRAPH ON THIS UNIT
                    ISAV1            1  FIRST ORDINATE PUNCHED OR SAVED
                    ISAV2          288  LAST ORDINATE PUNCHED OR SAVED
                   TIMINT         .083  TIME INTERVAL IN HOURS


             SUBBASIN RUNOFF DATA

157 BA         SUBBASIN CHARACTERISTICS
                    TAREA          .01  SUBBASIN AREA

               PRECIPITATION DATA

  8 PB              STORM         2.52  BASIN TOTAL PRECIPITATION

 10 PI           INCREMENTAL PRECIPITATION PATTERN
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .01      .01      .01      .01      .01      .01      .01      .01
                    .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                    .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                    .01      .01      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
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                    .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                    .00      .00      .00      .00      .00      .00      .00

158 LS         SCS LOSS RATE
                   STRTL           .44  INITIAL ABSTRACTION
                  CRVNBR         82.00  CURVE NUMBER
                   RTIMP           .00  PERCENT IMPERVIOUS AREA

159 UD         SCS DIMENSIONLESS UNITGRAPH
                    TLAG           .08  LAG

                                                               ***

                                                        UNIT HYDROGRAPH
                                                    7 END-OF-PERIOD ORDINATES
             44.       45.       14.        5.        2.        1.        0.

      ***             ***             ***             ***             ***

                      HYDROGRAPH AT STATION      SB40

   TOTAL RAINFALL =     2.52, TOTAL LOSS =     1.51, TOTAL EXCESS =     1.01

 PEAK FLOW      TIME                     MAXIMUM AVERAGE FLOW
                                    6-HR        24-HR        72-HR        23.92-HR
+   (CFS)       (HR)
                          (CFS)
+       3.     13.00                 1.          0.           0.            0.
                        (INCHES)   .681       1.009        1.009         1.009
                         (AC-FT)     1.          1.           1.            1.

                       CUMULATIVE AREA =      .01 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

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                  *          *
 '61 KK           *    HC24  *
                  *          *
                  ************

 163 KO               OUTPUT CONTROL VARIABLES
                           IPRNT          3  PRINT CONTROL
                           IPLOT          0  PLOT CONTROL
                           QSCAL         0.  HYDROGRAPH PLOT SCALE
                           IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                            IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                           ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                           ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                          TIMINT       .083  TIME INTERVAL IN HOURS

 162 HC               HYDROGRAPH COMBINATION
                           ICOMP          2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                                  ***

      ***            ***            ***            ***            ***

                              HYDROGRAPH AT STATION     HC24

 PEAK FLOW     TIME                              MAXIMUM AVERAGE FLOW
                                         6-HR        24-HR        72-HR      23.92-HR
+  (CFS)       (HR)
                              (CFS)
+     39.     13.00                       16.          6.           6.           6.
                            INCHES)      .680       1.006        1.006        1.006
                            (AC-FT)        8.         12.          12.          12.

                            CUMULATIVE AREA =      .22 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

                  **************
                  *            *
 164 KK           *    SB51    *
                  *            *
                  **************

 168 KO               OUTPUT CONTROL VARIABLES
                           IPRNT          3  PRINT CONTROL
                           IPLOT          0  PLOT CONTROL
                           QSCAL         0.  HYDROGRAPH PLOT SCALE
                           IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                            IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                           ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                           ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                          TIMINT       .083  TIME INTERVAL IN HOURS

                    SUBBASIN RUNOFF DATA

 165 BA               SUBBASIN CHARACTERISTICS
                           TAREA        .04  SUBBASIN AREA

                      PRECIPITATION DATA

   8 PB                    STORM       2.52  BASIN TOTAL PRECIPITATION

  10 PI                 INCREMENTAL PRECIPITATION PATTERN
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .01      .01      .01      .01      .01      .01      .01      .01
                           .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                           .01      .01      .01      .01      .01      .01      .01      .01      .01      .01
                           .01      .01      .00      .00      .00      .00      .00      00       .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
                           .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
```

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                    .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                    .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                    .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                    .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                    .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                    .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                    .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                    .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                    .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                    .00       .00       .00       .00       .00       .00       .00

166 LS          SCS LOSS RATE
                      STRTL         .44  INITIAL ABSTRACTION
                     CRVNBR       82.00  CURVE NUMBER
                      RTIMP         .00  PERCENT IMPERVIOUS AREA

167 UD          SCS DIMENSIONLESS UNITGRAPH
                       TLAG         .08  LAG

                                                          ***

                                                     UNIT HYDROGRAPH
                                                 7 END-OF-PERIOD ORDINATES
             130.       132.       43.        14.        5.        2.        0.

     ***             ***             ***             ***             ***

                          HYDROGRAPH AT STATION       SB51

  TOTAL RAINFALL =     2.52, TOTAL LOSS =     1.51, TOTAL EXCESS =     1.01

PEAK FLOW      TIME                           MAXIMUM AVERAGE FLOW
                                    6-HR         24-HR        72-HR       23.92-HR
+  (CFS)       (HR)
                           (CFS)
+      8.     13.00                    3.           1.           1.           1.
                        (INCHES)     .681        1.009        1.009        1.009
                         (AC FT)       2.           2.           2.           2.

                         CUMULATIVE AREA =       .04 SQ MI



* *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


            **************
            *            *
169 KK      *     CV29   *
            *            *
            **************

171 KO          OUTPUT CONTROL VARIABLES
                      IPRNT           3  PRINT CONTROL
                      IPLOT           0  PLOT CONTROL
                      QSCAL          0.  HYDROGRAPH PLOT SCALE
                      IPNCH           0  PUNCH COMPUTED HYDROGRAPH
                       IOUT          22  SAVE HYDROGRAPH ON THIS UNIT
                      ISAV1           1  FIRST ORDINATE PUNCHED OR SAVED
                      ISAV2         288  LAST ORDINATE PUNCHED OR SAVED
                     TIMINT        .083  TIME INTERVAL IN HOURS


              HYDROGRAPH ROUTING DATA

170 RD          MUSKINGUM-CUNGE CHANNEL ROUTING
                          L        324.  CHANNEL LENGTH
                          S       .2500  SLOPE
                          N        .030  CHANNEL ROUGHNESS COEFFICIENT
                         CA         .00  CONTRIBUTING AREA
                      SHAPE        CIRC  CHANNEL SHAPE
                         WD        1.50  BOTTOM WIDTH OR DIAMETER
                          Z         .00  SIDE SLOPE

                                                          ***
                                    COMPUTED MUSKINGUM-CUNGE PARAMETERS
                                           COMPUTATION TIME STEP
                  ELEMENT    ALPHA      M          DT        DX       PEAK     TIME TO     VOLUME     MAXIMUM
                                                                                 PEAK                 CELERITY
                                                 (MIN)      (FT)     (CFS)      (MIN)       (IN)      (FPS)

                    MAIN     14.34     1.25        .39     161.96     8.01      780.03      1.01       13.89

                                         INTERPOLATED TO SPECIFIED COMPUTATION INTERVAL
```

```
                    MAIN      14 34      1.25      5.00                 8.01    780.00     1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .2261E+01 EXCESS= .0000E+00 OUTFLOW= .2260E+01 BASIN STORAGE= .7173E-03 PERCENT ERROR=    .0


      ***          ***          ***          ***          ***

                       HYDROGRAPH AT STATION     CV29

   PEAK FLOW     TIME                        MAXIMUM AVERAGE FLOW
                                   6-HR       24-HR       72-HR      23.92-HR
 +   (CFS)       (HR)
                         (CFS)
 +       8.     13.00                3.          1.          1.           1.
                       (INCHES)    .681       1.009       1.009        1.009
                        (AC-FT)      2.          2.          2.           2.

                      CUMULATIVE AREA =     .04 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***


              **************
              *            *
  172 KK      *     SB52   *
              *            *
              **************

  176 KO           OUTPUT CONTROL VARIABLES
                         IPRNT          3  PRINT CONTROL
                         IPLOT          0  PLOT CONTROL
                         QSCAL         0.  HYDROGRAPH PLOT SCALE
                         IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                          IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                         ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                         ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                        TIMINT       .083  TIME INTERVAL IN HOURS


                 SUBBASIN RUNOFF DATA

  173 BA           SUBBASIN CHARACTERISTICS
                         TAREA        .02  SUBBASIN AREA

                   PRECIPITATION DATA

    8 PB                 STORM       2.52  BASIN TOTAL PRECIPITATION

   10 PI             INCREMENTAL PRECIPITATION PATTERN
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .01       .01       .01       .01       .01       .01       .01       .01
                          .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                          .01       .01       .01       .01       .01       .01       .01       .01       .01       .01
                          .01       .01       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00       .00       .00       .00
                          .00       .00       .00       .00       .00       .00       .00

  174 LS           SCS LOSS RATE
                        STRTL         .44  INITIAL ABSTRACTION
                        CRVNBR      82.00  CURVE NUMBER
```

```
                    RTIMP          .00  PERCENT IMPERVIOUS AREA

 75 UD           SCS DIMENSIONLESS UNITGRAPH
                     TLAG          .08  LAG

                                                           ***

                                                     UNIT HYDROGRAPH
                                                 7 END-OF-PERIOD ORDINATES
            65        66.       21.        7.         2.         1.         0.

     ***            ***           ***             ***             ***

                      HYDROGRAPH AT STATION     SB52

   TOTAL RAINFALL =    2.52, TOTAL LOSS =    1.51, TOTAL EXCESS =    1.01

  PEAK FLOW     TIME                           MAXIMUM AVERAGE FLOW
                                     6-HR        24-HR        72-HR      23.92-HR
+   (CFS)       HR)
                        (CFS)
+       4.     13.00                   2.           1.           1.            1.
                      (INCHES)       .681        1.009        1.009         1.009
                       (AC-FT)         1.           1.           1.            1.

                     CUMULATIVE AREA =     .02 SQ MI


*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***

           **************
           *            *
  177 KK   *     HC29   *
           *            *
           **************

  179 KO         OUTPUT CONTROL VARIABLES
                    IPRNT          3  PRINT CONTROL
                    IPLOT          0  PLOT CONTROL
                    QSCAL         0.  HYDROGRAPH PLOT SCALE
                    IPNCH          0  PUNCH COMPUTED HYDROGRAPH
                     IOUT         22  SAVE HYDROGRAPH ON THIS UNIT
                    ISAV1          1  FIRST ORDINATE PUNCHED OR SAVED
                    ISAV2        288  LAST ORDINATE PUNCHED OR SAVED
                   TIMINT       .083  TIME INTERVAL IN HOURS


  178 HC         HYDROGRAPH COMBINATION
                    ICOMP          2  NUMBER OF HYDROGRAPHS TO COMBINE

                                                           ***

     ***            ***           ***             ***             ***

                      HYDROGRAPH AT STATION     HC29

  PEAK FLOW     TIME                           MAXIMUM AVERAGE FLOW
                                     6-HR        24-HR        72-HR      23.92-HR
+   (CFS)       (HR)
                        (CFS)
+      12.     13.00                   5.           2.           2.            2.
                      (INCHES)       .681        1.009        1.009         1.009
                       (AC-FT)         2.           3.           3.            3.

                     CUMULATIVE AREA =     .06 SQ MI

1
                                                     RUNOFF SUMMARY
                                               FLOW IN CUBIC FEET PER SECOND
                                            TIME IN HOURS,  AREA IN SQUARE MILES

                                        PEAK    TIME OF      AVERAGE FLOW FOR MAXIMUM PERIOD        BASIN     MAXIMUM     TIME OF
          OPERATION        STATION      FLOW     PEAK                                                AREA      STAGE     MAX STAGE
+                                                            6-HOUR       24-HOUR       72-HOUR

          HYDROGRAPH AT
+                             SB7         19.    13.08          8.           3.            3.         .11

          ROUTED TO
+                             CV3         19.    13.08          8.           3.            3          .11

          HYDROGRAPH AT
                             SB19          3.    13.00          1.           0.            0.         .01
```

	2 COMBINED AT	HC3	21.	13.08	9.	3	3.	.12
+	HYDROGRAPH AT	SB20	3.	13.00	1.	0.	0.	.01
+	ROUTED TO	CV4	3.	13.00	1.	0.	0.	.01
+	2 COMBINED AT	HC4	23.	13.00	10.	4.	4.	.14
+	HYDROGRAPH AT	SB46	1.	13.00	1.	0.	0.	.01
+	ROUTED TO	CV32	1.	13.00	1.	0.	0.	.01
+	HYDROGRAPH AT	SB47	1.	13.00	0.	0.	0.	.01
+	2 COMBINED AT	HC33	2.	13.00	1.	0.	0.	.01
+	HYDROGRAPH AT	SB8	43.	13.08	19.	7.	7.	.25
+	ROUTED TO	CV8	43.	13.08	19.	7.	7.	.25
+	HYDROGRAPH AT	SB24	3.	13.00	1.	0.	0.	.02
+	2 COMBINED AT	HC8	45.	13.08	20.	7.	7.	.27
+	ROUTED TO	CV9	45.	13.08	20.	7.	7.	.27
+	HYDROGRAPH AT	SB25	3.	13.00	1.	0.	0.	.01
	2 COMBINED AT	HC9	48.	13.08	21.	8.	8.	.28
+	HYDROGRAPH AT	SB9	31.	13.08	13.	5.	5.	.18
+	ROUTED TO	CV13	31.	13.08	13.	5.	5.	.18
+	HYDROGRAPH AT	SB29	3.	13.00	1.	0.	0.	.02
+	2 COMBINED AT	HC13	33.	13.08	15.	5.	5.	.20
+	ROUTED TO	CV14	33.	13.08	15.	5.	5.	.20
+	HYDROGRAPH AT	SB30	3.	13.00	1.	0.	0.	.01
+	2 COMBINED AT	HC14	36.	13.08	16.	6.	6.	.21
+	HYDROGRAPH AT	SB10	24.	13.08	10.	4.	4.	.13
+	ROUTED TO	CV18	24.	13.08	10.	4.	4.	.13
+	HYDROGRAPH AT	SB34	3.	13.00	1.	0.	0.	.02
+	2 COMBINED AT	HC18	27.	13.00	11.	4.	4.	.15
+	ROUTED TO	CV19	27.	13.00	11.	4.	4.	.15
+	HYDROGRAPH AT	SB35	3.	13.00	1.	0.	0.	.01
	2 COMBINED AT	HC19	29.	13.00	12.	4.	4.	.16

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        HYDROGRAPH AT
                       SB11        33.   13.08          14.          5.          5.         .19

        ROUTED TO
+                      CV23        33.   13.08          14.          5.          5.         .19

        HYDROGRAPH AT
+                      SB39         3.   13.00           1.          0.          0.         .02

        2 COMBINED AT
+                      HC23        36.   13.00          15.          6.          6.         .20

        ROUTED TO
+                      CV24        36.   13.00          15.          6.          6.         .20

        HYDROGRAPH AT
+                      SB40         3.   13.00           1.          0.          0.         .01

        2 COMBINED AT
+                      HC24        39.   13.00          16.          6.          6.         .22

        HYDROGRAPH AT
+                      SB51         8.   13.00           3.          1.          1.         .04

        ROUTED TO
+                      CV29         8.   13.00           3.          1.          1.         .04

        HYDROGRAPH AT
+                      SB52         4.   13.00           2.          1.          1.         .02

        2 COMBINED AT
+                      HC29        12.   13.00           5.          2.          2.         .06
1
                          SUMMARY OF KINEMATIC WAVE - MUSKINGUM-CUNGE ROUTING
                              (FLOW IS DIRECT RUNOFF WITHOUT BASE FLOW)
                                                                 INTERPOLATED TO
                                                              COMPUTATION INTERVAL
          ISTAQ    ELEMENT     DT       PEAK    TIME TO    VOLUME      DT      PEAK    TIME TO     VOLUME
                                                  PEAK                                    PEAK

                              (MIN)     (CFS)     (MIN)      (IN)     (MIN)    (CFS)     (MIN)      (IN)

             CV3   MANE         .20     18.77    785.32      1.00      5.00    18.75    785.00       1.00

 NTINUITY SUMMARY (AC-FT) - INFLOW= .5945E+01 EXCESS= .0000E+00 OUTFLOW= .5944E+01 BASIN STORAGE= .9746E-03 PERCENT ERROR=     .0

             CV4   MANE         .32      2.56    780.16      1.01      5.00     2.55    780.00       1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .7215E+00 EXCESS= .0000E+00 OUTFLOW= .7213E+00 BASIN STORAGE= .1864E-03 PERCENT ERROR=     .0

            CV32   MANE         .44      1.33    779.97      1.01      5.00     1.33    780.00       1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .3769E+00 EXCESS= .0000E+00 OUTFLOW= .3767E+00 BASIN STORAGE= .1346E-03 PERCENT ERROR=     .0

             CV8   MANE         .17     43.03    785.29      1.00      5.00    42.97    785.00       1.00

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1363E+02 EXCESS= .0000E+00 OUTFLOW= .1363E+02 BASIN STORAGE= .1903E-02 PERCENT ERROR=     .0

             CV9   MANE         .17     45.42    785.13      1.00      5.00    45.38    785.00       1.00

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1447E+02 EXCESS= .0000E+00 OUTFLOW= .1447E+02 BASIN STORAGE= .2025E-02 PERCENT ERROR=     .0

            CV13   MANE         .19     30.86    785.29      1.00      5.00    30.83    785.00       1.00

CONTINUITY SUMMARY (AC-FT) - INFLOW= .9773E+01 EXCESS= .0000E+00 OUTFLOW= .9772E+01 BASIN STORAGE= .1482E-02 PERCENT ERROR=     .0

            CV14   MANE         .18     33.28    785.20      1.00      5.00    33.26    785.00       1.00

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1062E+02 EXCESS= .0000E+00 OUTFLOW= .1062E+02 BASIN STORAGE= .1546E-02 PERCENT ERROR=     .0

            CV18   MANE         .2      23.86    784.95      1.01      5.00    23.88    785.00       1.01
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```
CONTINUITY SUMMARY (AC-FT) - INFLOW= .7212E+01 EXCESS= .0000E+00 OUTFLOW= .7211E+01 BASIN STORAGE= .1151E-02 PERCENT ERROR=    .0

            CV19  MANE          .19     26.77    780.25      1.01      5.00     26.73    780.00       1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .8061E+01 EXCESS= .0000E+00 OUTFLOW= .8060E+01 BASIN STORAGE= .1268E-02 PERCENT ERROR=    .0

            CV23  MANE          .19     33.29    785.00      1.01      5.00     33.29    785.00       1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1005E+02 EXCESS= .0000E+00 OUTFLOW= .1005E+02 BASIN STORAGE= .1513E-02 PERCENT ERROR=    .0

            CV24  MANE          .18     36.09    780.25      1.01      5.00     36.04    780.00       1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .1089E+02 EXCESS= .0000E+00 OUTFLOW= .1089E+02 BASIN STORAGE= .1578E-02 PERCENT ERROR=    .0

            CV29  MANE          .39  -   8.01    780.03      1.01      5.00      8.01    780.00       1.01

CONTINUITY SUMMARY (AC-FT) - INFLOW= .2261E+01 EXCESS= .0000E+00 OUTFLOW= .2260E+01 BASIN STORAGE= .7173E-03 PERCENT ERROR=    .0


*** NORMAL END OF HEC-1 ***
```

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area [2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

1 Average runoff condition, and $I_a = 0.2S$.
2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
4 Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
5 Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

POINT PRECIPITATION FREQUENCY ESTIMATES FROM NOAA ATLAS 14

Utah 40.85579°N 112.75219°W 4435 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 1, Version 3
G.M. Bonnin, D. Todd, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2003
Extracted: Thu Nov 18 2004

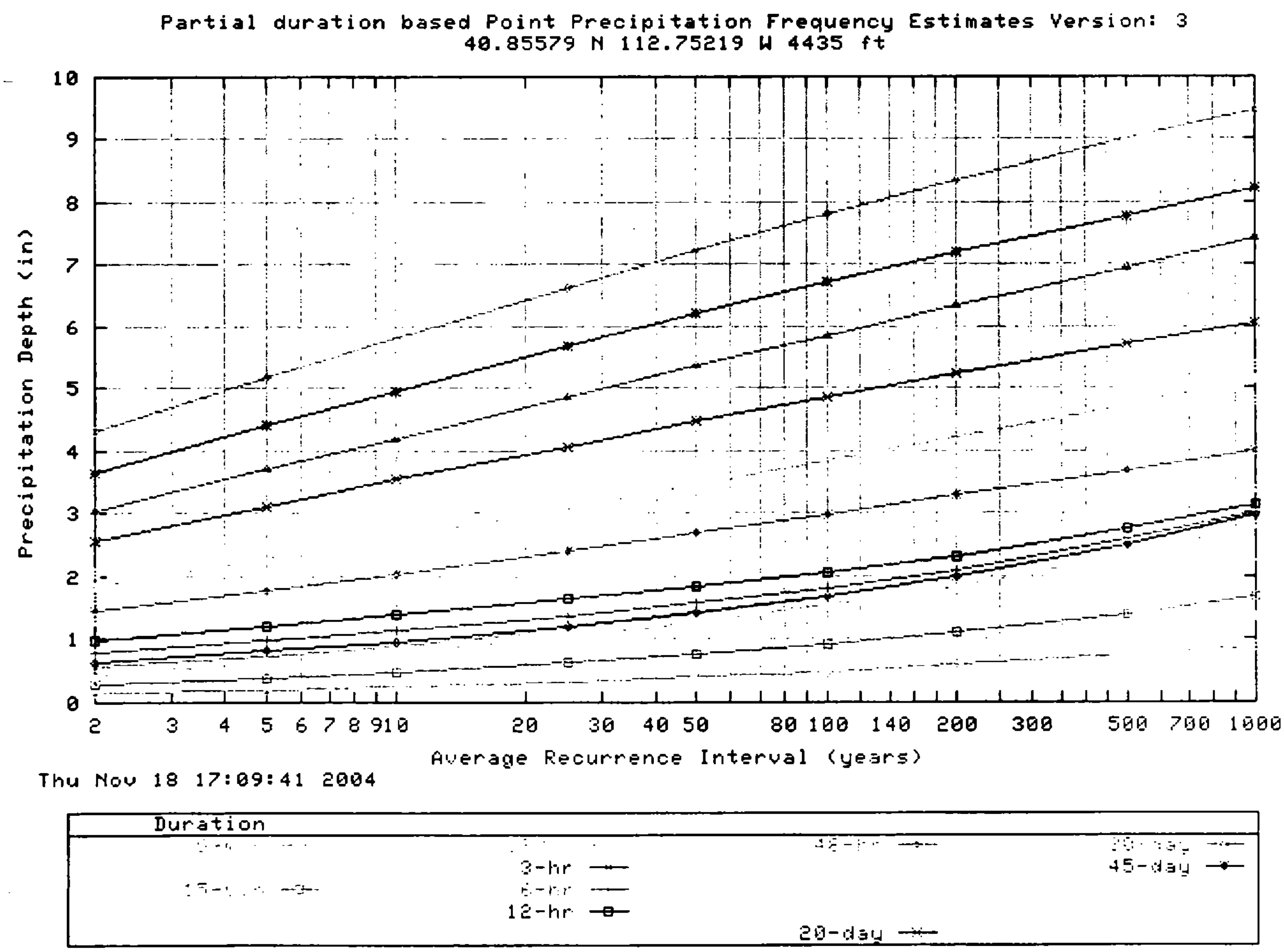
Confidence Limits | Seasonality | Location Maps | Other Info. | Grids | Maps | Help | Docs | U.S. M

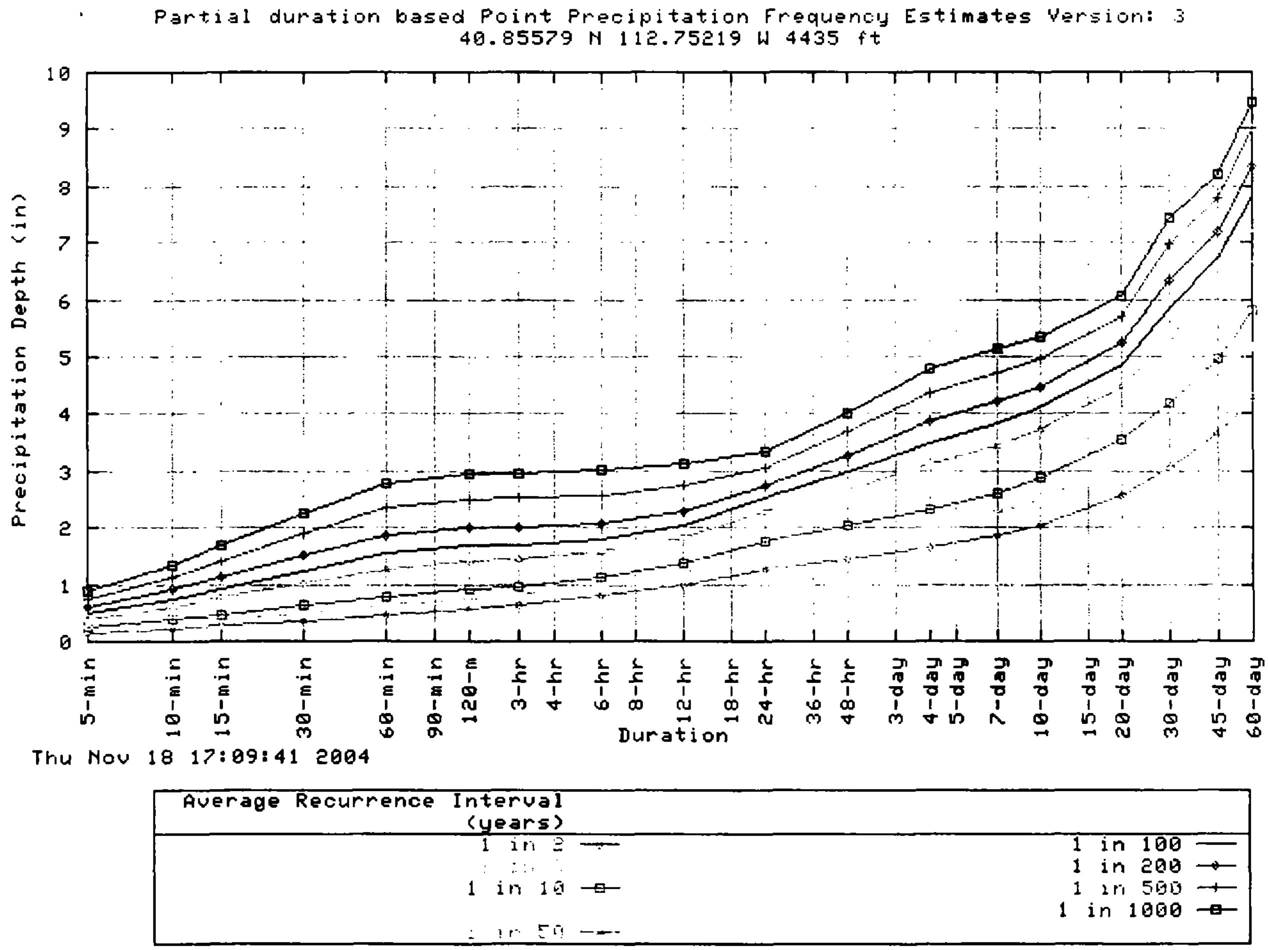
Precipitation Frequency Estimates (inches)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	0.14	0.22	0.27	0.37	0.45	0.56	0.63	0.80	0.99	1.27	1.45	1.64	1.85	2.06	2.58	3.05	3.67	4.30
5	0.20	0.30	0.38	0.51	0.63	0.74	0.81	0.98	1.21	1.54	1.77	2.02	2.28	2.51	3.12	3.70	4.41	5.16
10	0.25	0.38	0.47	0.64	0.79	0.90	0.96	1.14	1.38	1.76	2.04	2.33	2.62	2.87	3.54	4.21	4.97	5.81
25	0.33	0.51	0.63	0.84	1.04	1.16	1.21	1.38	1.64	2.06	2.40	2.77	3.09	3.36	4.08	4.87	5.69	6.64
50	0.40	0.62	0.76	1.03	1.27	1.40	1.43	1.57	1.84	2.29	2.69	3.12	3.45	3.73	4.47	5.37	6.21	7.23
100	0.49	0.75	0.93	1.25	1.54	1.67	1.70	1.80	2.06	2.52	2.98	3.48	3.83	4.11	4.87	5.86	6.72	7.81
200	0.59	0.90	1.11	1.50	1.86	1.99	2.02	2.09	2.31	2.75	3.29	3.86	4.21	4.49	5.24	6.34	7.19	8.34
500	0.75	1.14	1.41	1.90	2.35	2.50	2.52	2.59	2.75	3.08	3.70	4.38	4.73	4.98	5.72	6.96	7.79	9.01
1000	0.89	1.35	1.68	2.26	2.80	2.95	2.97	3.03	3.13	3.36	4.01	4.79	5.13	5.36	6.06	7.41	8.21	9.47

Text version of table

* These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval. Please refer to the documentation for more information. NOTE: Formatting forces estimates near zero to appear as zero.





Confidence Limits -

* Upper bound of the 90% confidence interval Precipitation Frequency Estimates (inches)																		
ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
2	0.17	0.25	0.32	0.42	0.53	0.63	0.71	0.87	1.08	1.41	1.61	1.81	2.06	2.29	2.84	3.37	4.04	4.75
5	0.23	0.35	0.43	0.59	0.72	0.83	0.90	1.07	1.31	1.72	1.97	2.23	2.52	2.79	3.45	4.10	4.86	5.70
10	0.29	0.44	0.55	0.73	0.91	1.01	1.07	1.25	1.50	1.96	2.26	2.58	2.91	3.19	3.91	4.66	5.47	6.42
25	0.38	0.58	0.72	0.97	1.21	1.31	1.36	1.51	1.79	2.29	2.67	3.06	3.43	3.73	4.50	5.39	6.26	7.32
50	0.47	0.72	0.89	1.20	1.48	1.59	1.64	1.75	2.02	2.55	2.99	3.45	3.83	4.16	4.94	5.95	6.84	7.99
100	0.58	0.88	1.09	1.47	1.82	1.93	1.97	2.06	2.30	2.82	3.33	3.85	4.26	4.58	5.38	6.50	7.41	8.64
200	0.71	1.07	1.33	1.79	2.22	2.34	2.38	2.46	2.62	3.09	3.68	4.29	4.69	5.01	5.81	7.05	7.95	9.25
500	0.91	1.39	1.72	2.32	2.87	3.01	3.05	3.12	3.19	3.47	4.16	4.89	5.30	5.61	6.36	7.79	8.64	10.03
1000	1.10	1.68	2.09	2.81	3.48	3.62	3.67	3.73	3.78	3.80	4.54	5.38	5.78	6.05	6.78	8.34	9.14	10.58

* The upper bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are greater than.
** These precipitation frequency estimates are based on a partial duration series. ARI is the Average Recurrence Interval.
Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

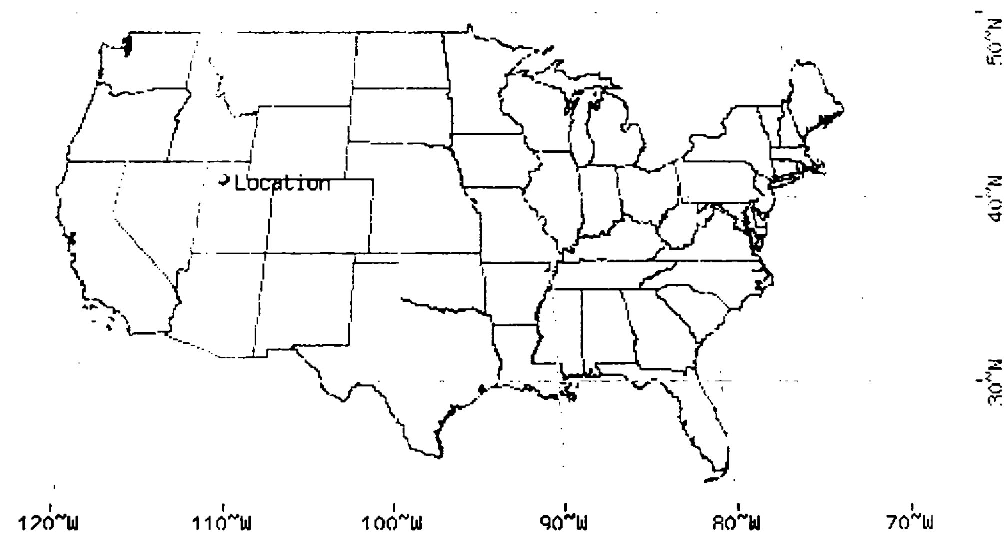
* Lower bound of the 90% confidence interval Precipitation Frequency Estimates (inches)																		
ARI** (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day

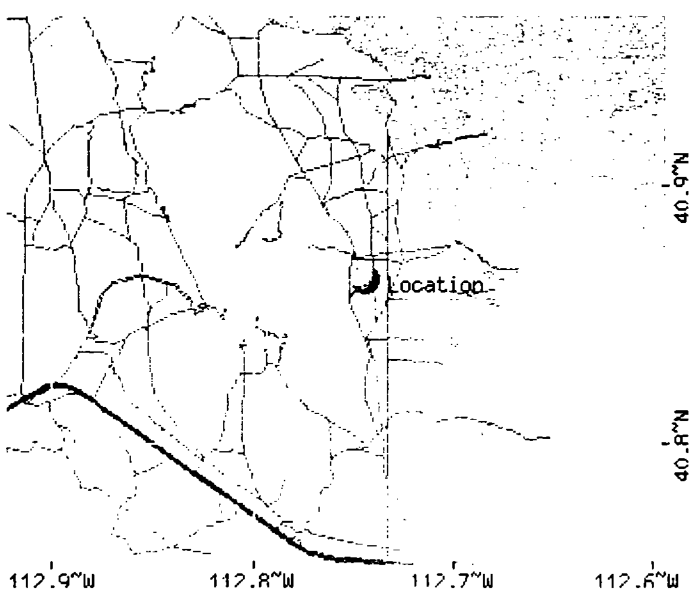
2	0.13	0.19	0.24	0.32	0.40	0.50	0.58	0.74	0.92	1.14	1.31	1.49	1.68	1.86	2.34	2.76	3.34	3.90
5	0.17	0.27	0.33	0.45	0.55	0.66	0.74	0.91	1.12	1.39	1.60	1.83	2.06	2.27	2.83	3.35	4.01	4.67
10	0.22	0.33	0.41	0.55	0.69	0.80	0.87	1.05	1.28	1.59	1.84	2.12	2.37	2.59	3.20	3.80	4.52	5.26
25	0.28	0.43	0.53	0.71	0.89	1.01	1.07	1.25	1.50	1.85	2.16	2.50	2.79	3.01	3.68	4.39	5.16	5.98
50	0.34	0.51	0.63	0.85	1.05	1.19	1.24	1.41	1.67	2.05	2.40	2.79	3.09	3.34	4.03	4.82	5.62	6.50
100	0.40	0.60	0.75	1.00	1.24	1.38	1.43	1.58	1.83	2.24	2.65	3.10	3.41	3.65	4.37	5.24	6.06	6.99
200	0.46	0.70	0.87	1.17	1.45	1.59	1.66	1.80	2.02	2.43	2.90	3.40	3.73	3.96	4.68	5.64	6.47	7.45
500	0.55	0.84	1.04	1.41	1.74	1.89	1.98	2.16	2.35	2.69	3.22	3.82	4.14	4.36	5.08	6.14	6.96	8.00
1000	0.63	0.96	1.20	1.61	1.99	2.15	2.25	2.46	2.62	2.91	3.47	4.13	4.45	4.65	5.36	6.50	7.31	8.38

* The lower bound of the confidence interval at 90% confidence level is the value which 5% of the simulated quantile values for a given frequency are less than.
** These precipitation frequency estimates are based on a partial duration maxima series. ARI is the Average Recurrence Interval.

Please refer to the documentation for more information. NOTE: Formatting prevents estimates near zero to appear as zero.

Maps -





These maps were produced using a direct map request from the U.S. Census Bureau Mapping and Cartographic Resources Tiger Map Server.

Please read disclaimer for more information.

LEGEND

State
County
Indian Resv
Lake/Pond/Ocean
Street
Expressway
Highway
Connector
Stream
Military Area
National Park
Other Park
City
County

Scale 1:228583 0 2 4 6 8 mi / 0 2 4 6 8 10 km
*average--true scale depends on monitor resolution

Other Maps/Photographs -

View USGS digital orthophoto quadrangle (DOQ) covering this location from TerraServer; **USGS Aerial Photograph** may also be available from this site. A DOQ is a computer-generated image of an aerial photograph in which image displacement caused by terrain relief and camera tilts has been removed. It combines the image characteristics of a photograph with the geometric qualities of a map. Visit the USGS for more information.

..atershed/Stream Flow Information -

Find the Watershed for this location using the U.S. Environmental Protection Agency's site.

Climate Data Sources -

Precipitation frequency results are based on data from a variety of sources, but largely NCDC. The following links provide general information
about observing sites in the area, regardless of if their data was used in this study. For detailed information about the stations used in this study,
please refer to our documentation.

Using the **National Climatic Data Center's** (NCDC) station search engine, locate other climate stations within:

+/-30 minutes ...OR... +/-1 degree of this location (40.85579/-112.75219). Digital ASCII data can be obtained directly from **NCDC**.

Find Natural Resources Conservation Service (NRCS) SNOTEL (SNOwpack TELemetry) stations by visiting the **Western Regional Climate Center's state-specific SNOTEL station maps.**

Hydrometeorological Design Studies Center
DOC/NOAA/National Weather Service
1325 East-West Highway
"ver Spring, MD 20910
.) 713-1669
Questions?: HDSC.Questions@noaa.gov

Chapter 3 Time of Concentration and Travel Time

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system.

T_c influences the shape and peak of the runoff hydrograph. Urbanization usually decreases T_c, thereby increasing the peak discharge. But T_c can be increased as a result of (a) ponding behind small or inadequate drainage systems, including storm drain inlets and road culverts, or (b) reduction of land slope through grading.

Factors affecting time of concentration and travel time

Surface roughness

One of the most significant effects of urban development on flow velocity is less retardance to flow. That is, undeveloped areas with very slow and shallow overland flow through vegetation become modified by urban development: the flow is then delivered to streets, gutters, and storm sewers that transport runoff downstream more rapidly. Travel time through the watershed is generally decreased.

Channel shape and flow patterns

In small non-urban watersheds, much of the travel time results from overland flow in upstream areas. Typically, urbanization reduces overland flow lengths by conveying storm runoff into a channel as soon as possible. Since channel designs have efficient hydraulic characteristics, runoff flow velocity increases and travel time decreases.

Slope

Slopes may be increased or decreased by urbanization, depending on the extent of site grading or the extent to which storm sewers and street ditches are used in the design of the water management system. Slope will tend to increase when channels are straightened and decrease when overland flow is directed through storm sewers, street gutters, and diversions.

Computation of travel time and time of concentration

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type that occurs is a function of the conveyance system and is best determined by field inspection.

Travel time (T_t) is the ratio of flow length to flow velocity:

$$T_t = \frac{L}{3600V} \qquad \text{[eq. 3-1]}$$

where:

T_t = travel time (hr)
L = flow length (ft)
V = average velocity (ft/s)
3600 = conversion factor from seconds to hours.

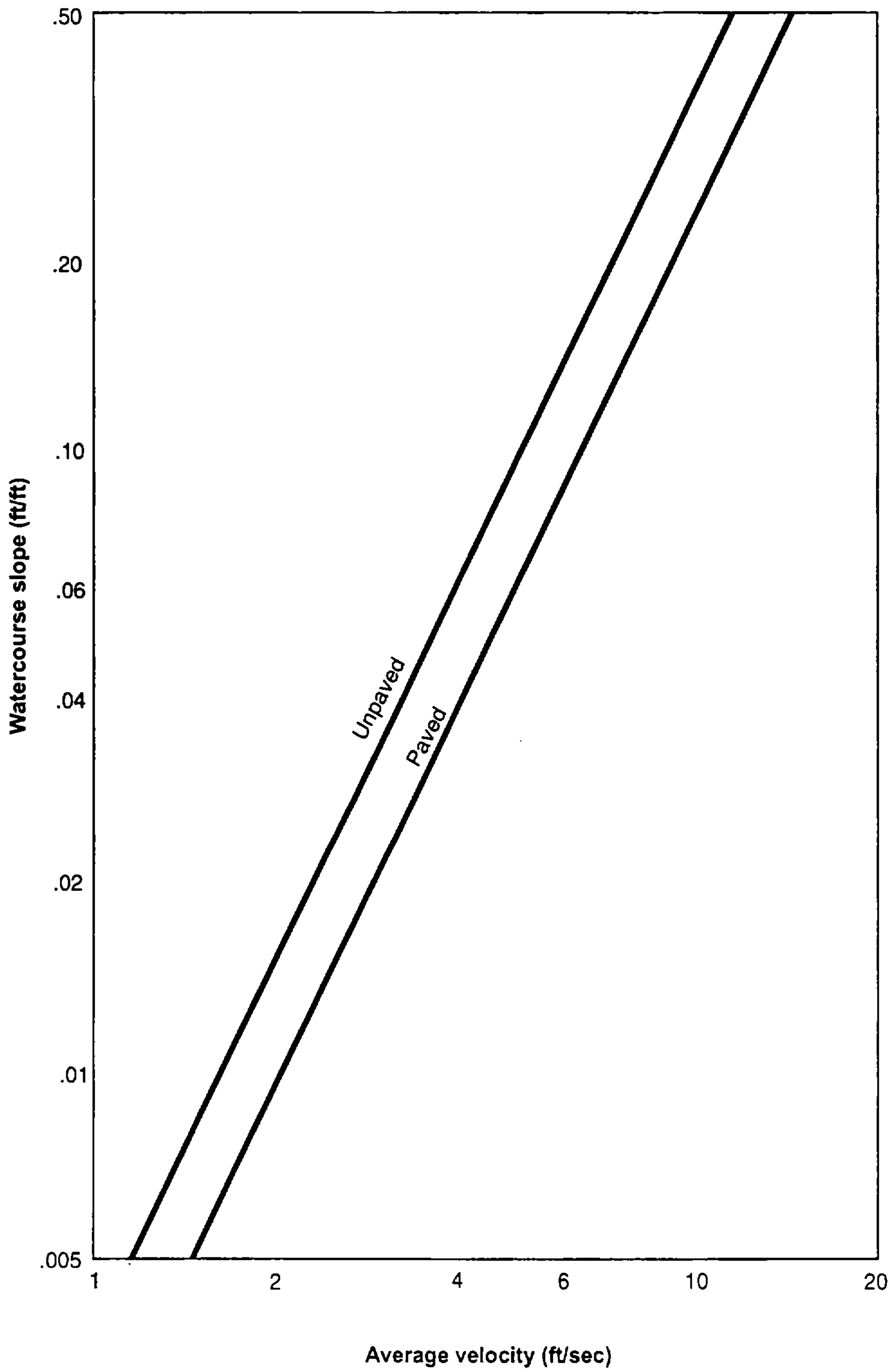
Time of concentration (T_c) is the sum of T_t values for the various consecutive flow segments:

$$T_c = T_{t_1} + T_{t_2} + \ldots T_{t_m} \qquad \text{[eq. 3-2]}$$

where:

T_c = time of concentration (hr)
m = number of flow segments

Figure 3-1 Average velocities for estimating travel time for shallow concentrated flow



Sheet flow

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value (Manning's n) is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such as litter, crop ridges, and rocks; and erosion and transportation of sediment. These n values are for very shallow flow depths of about 0.1 foot or so. Table 3-1 gives Manning's n values for sheet flow for various surface conditions.

Table 3-1 Roughness coefficients (Manning's n) for sheet flow

Surface description	n [1]
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods: [3]	
Light underbrush	0.40
Dense underbrush	0.80

1 The n values are a composite of information compiled by Engman (1986).
2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
3 When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

For sheet flow of less than 300 feet, use Manning's kinematic solution (Overtop and Meadows 1976) to compute T_t:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}\, s^{0.4}} \qquad \text{[eq. 3-3]}$$

where:

T_t = travel time (hr),
n = Manning's roughness coefficient (table 3-1)
L = flow length (ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (land slope, ft/ft)

This simplified form of the Manning's kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

Shallow concentrated flow

After a maximum of 300 feet, sheet flow usually becomes shallow concentrated flow. The average velocity for this flow can be determined from figure 3-1, in which average velocity is a function of watercourse slope and type of channel. For slopes less than 0.005 ft/ft, use equations given in appendix F for figure 3-1. Tillage can affect the direction of shallow concentrated flow. Flow may not always be directly down the watershed slope if tillage runs across the slope.

After determining average velocity in figure 3-1, use equation 3-1 to estimate travel time for the shallow concentrated flow segment.

Open channels

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines (indicating streams) appear on United States Geological Survey (USGS) quadrangle sheets. Manning's equation or water surface profile information can be used to estimate average flow velocity. Average flow velocity is usually determined for bankfull elevation.

Manning's equation is:

$$V = \frac{1.49 r^{\frac{2}{3}} s^{\frac{1}{2}}}{n} \qquad \text{[eq. 3-4]}$$

where:

- V = average velocity (ft/s)
- r = hydraulic radius (ft) and is equal to a/p_w
  - a = cross sectional flow area (ft²)
  - p_w = wetted perimeter (ft)
- s = slope of the hydraulic grade line (channel slope, ft/ft)
- n = Manning's roughness coefficient for open channel flow.

Manning's n values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using equation 3-4, T_t for the channel segment can be estimated using equation 3-1.

Reservoirs or lakes

Sometimes it is necessary to estimate the velocity of flow through a reservoir or lake at the outlet of a watershed. This travel time is normally very small and can be assumed as zero.

Limitations

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet. Equation 3-3 was developed for use with the four standard rainfall intensity-duration relationships.
- In watersheds with storm sewers, carefully identify the appropriate hydraulic flow path to estimate T_c. Storm sewers generally handle only a small portion of a large event. The rest of the peak flow travels by streets, lawns, and so on, to the outlet. Consult a standard hydraulics textbook to determine average velocity in pipes for either pressure or nonpressure flow.
- The minimum T_c used in TR-55 is 0.1 hour.
- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. The procedures in TR-55 can be used to determine the peak flow upstream of the culvert. Detailed storage routing procedures should be used to determine the outflow through the culvert.

Example 3-1

The sketch below shows a watershed in Dyer County, northwestern Tennessee. The problem is to compute T_c at the outlet of the watershed (point D). The 2-year 24-hour rainfall depth is 3.6 inches. All three types of flow occur from the hydraulically most distant point (A) to the point of interest (D). To compute T_c, first determine T_t for each segment from the following information:

Segment AB: Sheet flow; dense grass; slope (s) = 0.01 ft/ft; and length (L) = 100 ft. Segment BC: Shallow concentrated flow; unpaved; s = 0.01 ft/ft; and L = 1,400 ft. Segment CD: Channel flow; Manning's n = .05; flow area (a) = 27 ft²; wetted perimeter (p_w) = 28.2 ft; s = 0.005 ft/ft; and L = 7,300 ft.

See figure 3-2 for the computations made on worksheet 3.

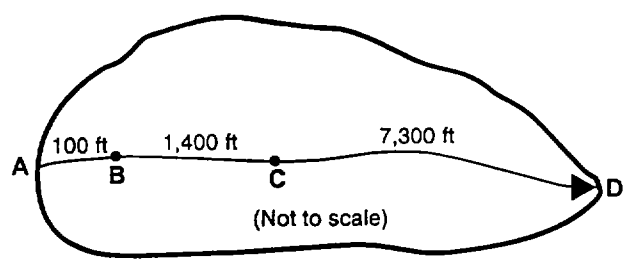


Figure 3-2 Worksheet 3 for example 3-1

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project *Heavenly Acres*	By *DW*	Date *10/6/85*
Location *Dyer County, Tennessee*	Checked *NM*	Date *10/8/85*

Check one: ☐ Present ☒ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Segment ID	AB		
1. Surface description (table 3-1)	Dense Grass		
2. Manning's roughness coefficient, n (table 3-1)	0.24		
3. Flow length, L (total L ≤ 300 ft) ... ft	100		
4. Two-year 24-hour rainfall, P_2 ... in	3.6		
5. Land slope, s ... ft/ft	0.01		
6. $T_t = \dfrac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t ... hr	0.30	+	= 0.30

Shallow concentrated flow

Segment ID	BC		
7. Surface description (paved or unpaved)	Unpaved		
8. Flow length, L ... ft	1400		
9. Watercourse slope, s ... ft/ft	0.01		
10. Average velocity, V (figure 3-1) ... ft/s	1.6		
11. $T_t = \dfrac{L}{3600\ V}$ Compute T_t ... hr	0.24	+	= 0.24

Segement ID	CD		
12. Cross sectional flow area, a ... ft^2	27		
13. Wetted perimeter, p_w ... ft	28.2		
14. Hydraulic radius, $r = \dfrac{a}{p_w}$ Compute r ... ft	0.957		
15 Channel slope, s ... ft/ft	0.005		
16. Manning's roughness coefficient, n ...	0.05		
17. $V = \dfrac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ... ft/s	2.05		
18. Flow length, L ... ft	7300		
19. $T_t = \dfrac{L}{3600\ V}$ Compute T_t ... hr	0.99	+	= 0.99
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) ... Hr			1.53

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project	By	Date
Wasatch Regional-Runoff	Gordon Jones	11/19/04
Location	**Checked**	**Date**
SB7, SB8, SB9		

Check one: ☒ Present ☐ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

	Segment ID			
1. Surface description (table 3-1)	Smooth Surface			
2. Manning's roughness coefficient, n (table 3-1)	.011			
3. Flow length, L (total L † 300 ft) ... ft	300			
4. Two-year 24-hour rainfall, P_2 ... in	1.27			
5. Land slope, s ... ft/ft	.05			
6. $T_t = \dfrac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t ... hr	.05	+		= .05

Shallow concentrated flow

	Segment ID			
7. Surface description (paved or unpaved)	Unpaved			
8. Flow length, L ... ft	3600			
9. Watercourse slope, s ... ft/ft	.05			
10. Average velocity, V (figure 3-1) ... ft/s	3.6			
11. $T_t = \dfrac{L}{3600\ V}$ Compute T_t ... hr	.28	+		= .28

[illegible]

	Segment ID			
12. Cross sectional flow area, a ... ft^2	3			
13. Wetted perimeter, p_w ... ft	6.32			
14. Hydraulic radius, $r = \dfrac{a}{p_w}$ Compute r ... ft	.474			
15 Channel slope, s ... ft/ft	.01			
16. Manning's roughness coefficient, n ...	.04			
17. $V = \dfrac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ... ft/s	2.26			
18. Flow length, L ... ft	1000			
19. $T_t = \dfrac{L}{3600\ V}$ Compute T_t ... hr	.12	+		= .12

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) ... Hr .45

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project: Wasatch Regional - Runoff	By: Gordon Jones	Date: 11/19/04
Location: All Slope SB	Checked	Date

Check one: ☒ Present ☐ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet.
Include a map, schematic, or description of flow segments.

Segment ID		
1. Surface description (table 3-1)	Smooth	
2. Manning's roughness coefficient, n (table 3-1)	.011	
3. Flow length, L (total L † 300 ft) .... ft	200	
4. Two-year 24-hour rainfall, P_2 .... in	1.27	
5. Land slope, s .... ft/ft	.25	

6. $T_t = \dfrac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t .... hr | .02 | + | | = | .02 |

Shallow concentrated flow

Segment ID		
7. Surface description (paved or unpaved)		
8. Flow length, L ....ft		
9. Watercourse slope, s .... ft/ft		
10. Average velocity, V (figure 3-1) .... ft/s		

11. $T_t = \dfrac{L}{3600\ V}$ Compute T_t .... hr | | + | | = | |

Segment ID		
12. Cross sectional flow area, a .... ft^2	3	
13. Wetted perimeter, p_w .... ft	6.32	
14. Hydraulic radius, $r = \dfrac{a}{p_w}$ Compute r .... ft	.474	
15 Channel slope, s .... ft/ft	.01	
16. Manning's roughness coefficient, n	.04	
17. $V = \dfrac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ....ft/s	2.26	
18. ~~Flow length~~, L .... ft	1000	

19. $T_t = \dfrac{L}{3600\ V}$ Compute T_t .... hr | .12 | + | | = | .12 |

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) .... Hr | .14 |

Worksheet 3: Time of Concentration (T_c) or travel time (T_t)

Project: Wasatch Regional - Bluff	By: Gordon Jones	Date: 11/19/04
Location: SB10, SB11	Checked	Date

Check one: ☒ Present ☐ Developed

Check one: ☒ T_c ☐ T_t through subarea

Notes: Space for as many as two segments per flow type can be used for each worksheet. Include a map, schematic, or description of flow segments.

[illegible]

	Segment ID			
1. Surface description (table 3-1)	Smooth			
2. Manning's roughness coefficient, n (table 3-1)	.011			
3. Flow length, L (total L † 300 ft) ... ft	300			
4. Two-year 24-hour rainfall, P_2 ... in	1.27			
5. Land slope, s ... ft/ft	.05			
6. $T_t = \frac{0.007\ (nL)^{0.8}}{P_2^{\ 0.5}\ s^{0.4}}$ Compute T_t ... hr	.05	+		= .05

Shallow concentrated flow

	Segment ID			
7. Surface description (paved or unpaved)	Unpaved			
8. Flow length, L ... ft	2000			
9. Watercourse slope, s ... ft/ft	.05			
10. Average velocity, V (figure 3-1) ... ft/s	3.6			
11. $T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	.15	+		= .15

[illegible]

	Segment ID			
12. Cross sectional flow area, a ... ft^2	3			
13. Wetted perimeter, p_w ... ft	6.32			
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r ... ft	.474			
15 Channel slope, s ... ft/ft	.01			
16. Manning's roughness coefficient, n	.04			
17. $V = \frac{1.49\ r^{2/3}\ s^{1/2}}{n}$ Compute V ... ft/s	2.26			
18. ~~Flow length~~, L ... ft	1000			
19. $T_t = \frac{L}{3600\ V}$ Compute T_t ... hr	.12	+		= .12

20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19) ... Hr .32

CLOSURE HYDRAULIC D

Purpose: To design the down spout to convey runoff from the closure cap.

Top of Cap Downspouts

Assumption:

- Assume parallel 24 inch pipes
- The maximum value of 21.5 cfs will be used for each pipe as the design criteria

Results: Manning's n = 0.024 ("ADS Specifier Manual - Civil Engineer", Advanced Drainage Systems, Inc.)

$Q = (1.49/0.024)(pi(12/12ft)^2)((12/12)/2)^{2\backslash 3}(0.25)^{0.5}$
$Q = 61.4\ ft^3/sec$

The 24 inch pipe is capable of carrying the maximum projected flows.

Chart 5 from the "US Department of Transportation Hydraulic Charts for the Selection of Highway Culverts" was used to determine required headwater depth. The required headwater depth is 3.0 ft for the design maximum flow of 21.5 cfs.

Side Slope Downspouts

Assumption:

- Assume 15 inch pipe
- The maximum value of 6 cfs will be used as the design criteria

Results: Manning's n = 0.024 ("ADS Specifier Manual - Civil Engineer", Advanced Drainage Systems, Inc.)

$Q = (1.49/0.024)(pi(7.5/12ft)^2)((7.5/12)/2)^{2\backslash 3}(0.25)^{0.5}$
$Q = 17.5\ ft^3/sec$

The 15 inch pipe is capable of carrying the maximum projected flows.

Chart 5 from the "US Department of Transportation Hydraulic Charts for the Selection of Highway Culverts" was used to determine required headwater depth. The required headwater depth is 21 inches (1.75 ft) for the design maximum flow of 6 cfs.

Southern Side Slope Downspouts

Assumption:

- Assume 24 inch pipe
- The maximum value of 12 cfs will be used as the design criteria

Results:

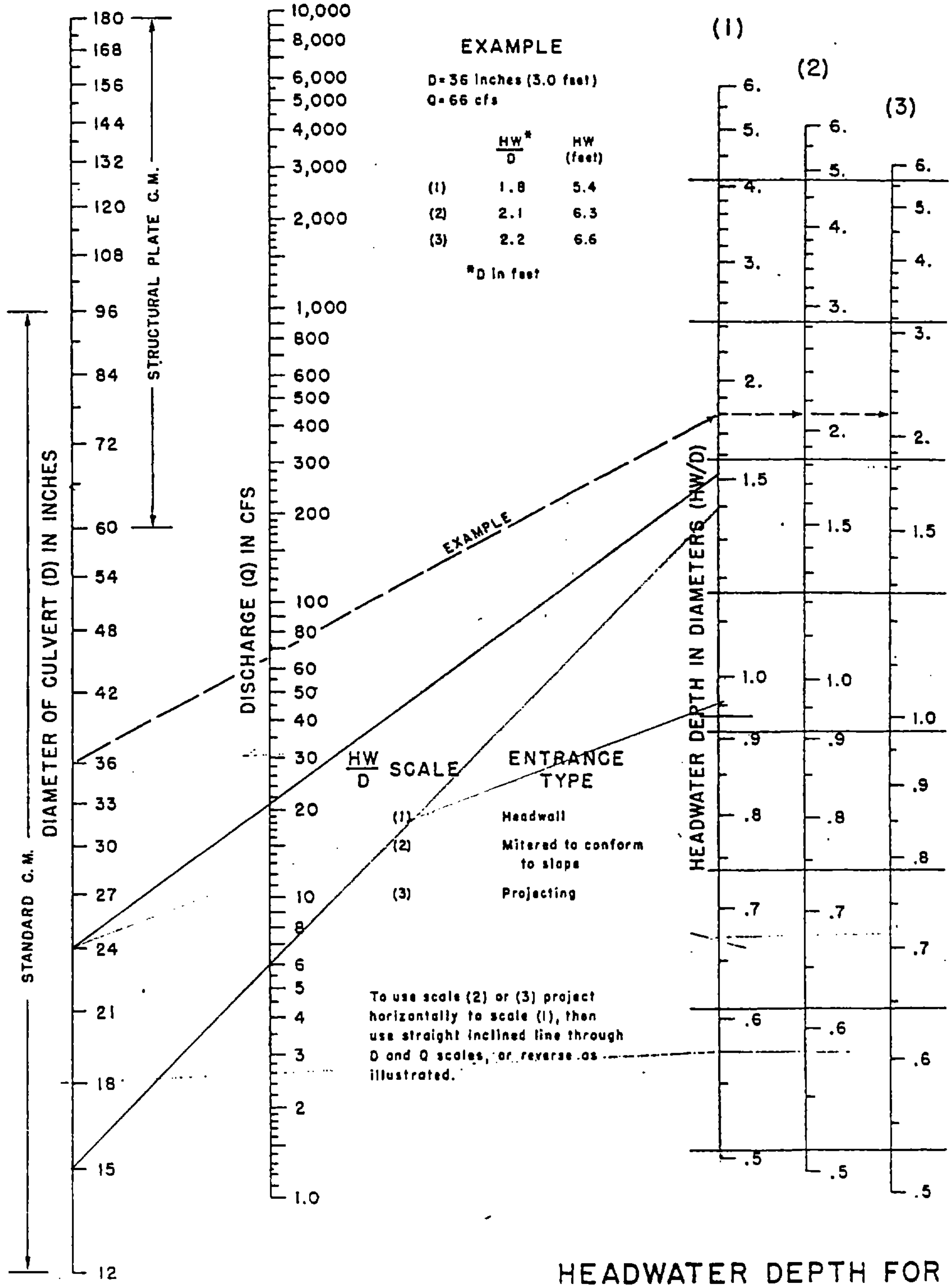
Manning's n = 0.024 ("ADS Specifier Manual - Civil Engineer", Advanced Drainage Systems, Inc.)

$Q = (1.49/0.024)(pi(12/12ft)^2)((12/12)/2)^{2/3}(0.25)^{0.5}$
$Q = 61.4\ ft^3/sec$

The 24 inch pipe is capable of carrying the maximum projected flows.

Chart 5 from the "US Department of Transportation Hydraulic Charts for the Selection of Highway Culverts" was used to determine required headwater depth. The required headwater depth is 1.9 ft for the design maximum flow of 12 cfs.

CHART 5



HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH INLET CONTROL

+ 10
BUREAU OF PUBLIC ROADS JAN. 1963

HANSEN ALLEN & LUCE inc ENGINEERS			
CLIENT:	Allied Waste	SHEET 1	OF 1
PROJECT:	Wasatch Regional	COMPUTED:	KCS
FEATURE:	Closure Sub Drainage Pipes	CHECKED:	
PROJECT NO.:	113.30.100	DATE: December 2004	

Purpose: To check the capacity requirements for sub-surface drainage pipes for the closure cap.

Required: In order to determine flows that may be contributing to the sub-surface drainage pipes, the following data is needed.

- Definition of flow contributing to the pipes.
- Hydraulic conductivity of the cover soil material.

Calculations:

Flow to the pipes depends on the capacity of the cover soil material to provide drainage to the pipes. The hydraulic conductivity of the soil is assumed to be about $5.2x10^{-4}$ cm/sec which is the default parameter used in the HELP model and appears to be representative of the soil types located at the facility.

Flow within the cover soil can be represented by to Darcy's law:

$q = KIA$ Where:

- q = Flow per unit width of the soil along the pipe, assume a unit width to be one foot.
- K = Hydraulic Conductivity $5.2x10^{-4}$ cm/sec = $1.706x10^{-5}$ ft/sec
- I = Hydraulic Gradient, 5% (0.05 ft/ft) for the closure cap surface.
- A = Area perpendicular to the flow path, 2 ft^2 (assume 1 ft wide by 2 feet thick)

Therefore:

$q = (1.706x10\text{-}5)*(0.05)*(2) = 1.706x10^{-6}$ cfs/ft of width.

Flow into the sub-drain pipe is assumed to be equivalent to the flow capable of moving through the cover soil material. Therefore:

$Q = qL$ Where:

- Q = Flow in sub-drainage pipe, cfs.
- L = Length of pipe receiving flow from the soil, ft.

The pipes follow the berms along the top of the east closure cap surface which are approximately 976 feet long on a slope of 1.0%. Therefore the flow in the pipe is:

Q = 1.706x13-6 cfs/ft (976 feet) = 0.0017 cfs or 0.012 gpm.

Capacity of 3 inch corrugated polyethylene pipe on a 1% slope = 0.05 cfs

Four sub-drainage pipes will tie into a single down drain giving 0.05 cfs for the down drain from the top surface of the closure cap.

Conclusions:

- 3-inch diameter CPE Pipe has sufficient capacity for the sub-drainage system.

HANSEN ALLEN & LUCE INC
ENGINEERS

CLIENT:	Allied Waste	SHEET 1	OF 1
PROJECT:	Wasatch Regional	COMPUTED:	KCS
FEATURE:	Storm Water Basin Inlet Pipes	CHECKED:	
PROJECT NO.:	113.30.100	DATE: December 2004	

Purpose: To determine the size of pipes that would be required under the facility road to the storm water basin for the design flows from around the landfill area.

Required: In order to determine pipe sizes, flows from the HEC-1 model need to be obtained.

Calculations:

Flows generated from the HEC-1 model are:

551 cfs for the southwest basin inlet, flows around the south end of the landfill area.
86 cfs for the northwest basin inlet, flows around the north end fo the landfill area.
48 cfs max for combined flows continuing from closure downspouts.

Pipe sizes are selected using nomographs provided in "hydraulic Charts for the Selection of Highway Culverts. It is assumed that inlet conditions control the pipe capacity since the pipes will have relatively steep slopes and the basin is of sufficient size that water will spread out quickly with very little ponding.

Divide the 551 cfs flow from around the south end of the landfill area by three and assume installation of 3 pipes in parallel. This results in 184 cfs per pipe.

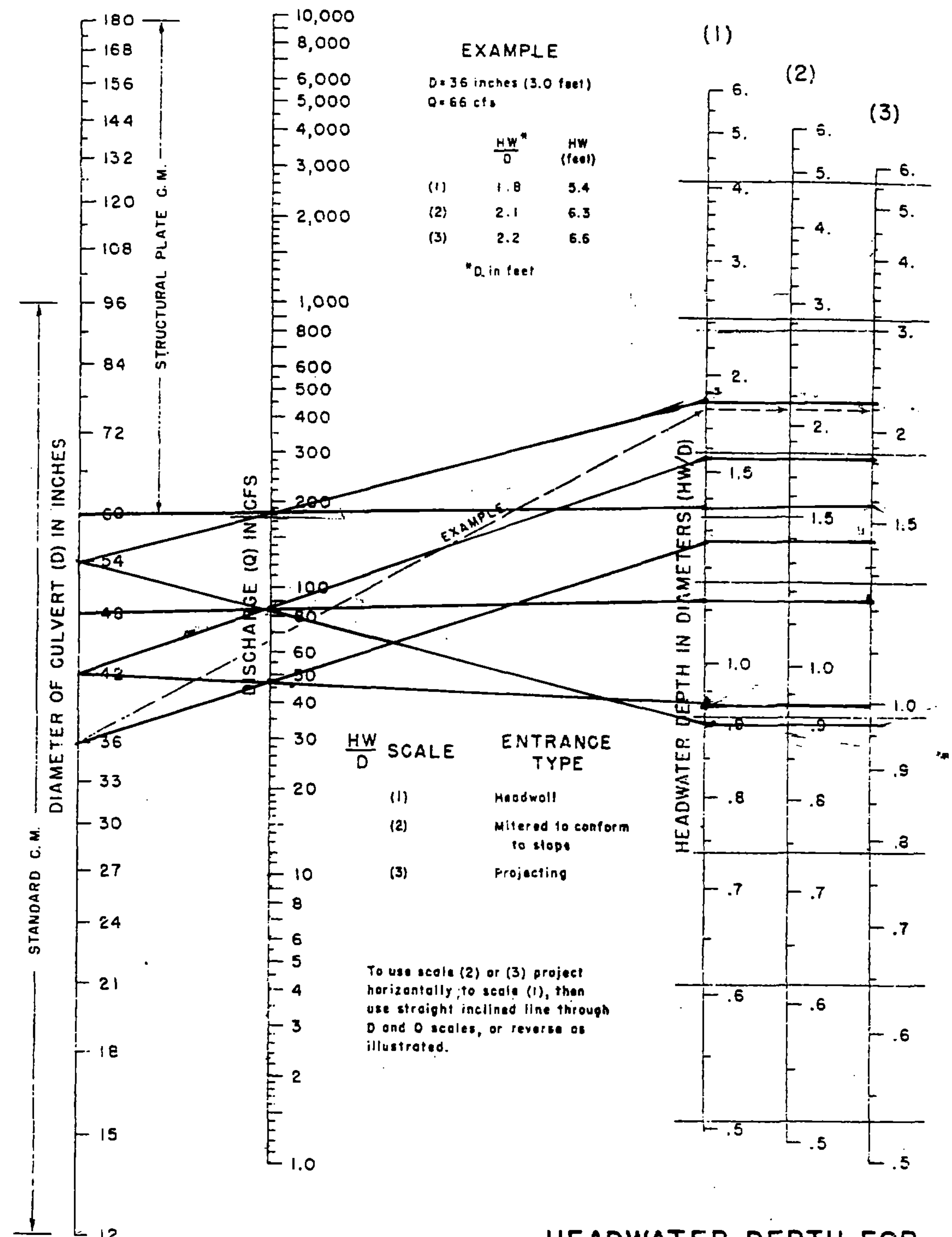
Conclusions:

Three 60-inch diameter pipes in parallel with 7.75 feet of head water depth will provide the flow capacity required for the 551 cfs.

One 48-inch diameter pipe with 4.8 feet of head water depth or one 54-inch diameter pipe with 4.3 feet of head water depth will provide the flow capacity required for the 86 cfs.

One 36-inch diameter pipe with 4.2 feet of head water depth will provide the flow capacity required for 48 cfs.

CHART 5



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

CLOSURE EROSION PRO

I. Purpose and Procedure.

The purpose of these calculations is determine which erosion protection measure to use and how to apply it. The closure cap will consist of a 4H:1V slope extending up from the top of the cell embankments. The embankments will consist of a 3H:1V slope from the top of the embankment down to the ground surface. The top of the closure cap will have a 5% slope. There will be a 5% section between the berm on the closure top that will combine with the 4H:1V slope.

The procedure used to determine the allowable slope lengths between the bench areas of the closure cap slopes is taken from the publication "Erosion and Sedimentation in Utah - A Guide for Control", Utah Water Research Laboratory, February 1984. This publication is specific to Utah. The figure presented on Sheet 2 presents a cross-section showing the configuration of the area contributing runoff to the slopes of the closure cap. Each slope between bench areas will consist of relatively uniform lengths such that the calculations for one slope length will be representative for each slope segment between benches along the slopes of the closure cap.

II. The procedure from the above publication uses the Universal Soil Loss Equation (in modified form to represent Utah's climatic and topographic conditions) to estimate the soil erosion potential of the surface soils assuming no application of erosion control measures. Erosion control measures to be implemented are based on the soil erosion potential calculated.

The universal soil loss equation used to calculate soil erosion potential is:

$$A = R \cdot K \cdot LS$$

where;

A = Computed amount of soil loss per unit area for the time interval represented by factor R, generally in tons per acre per year.

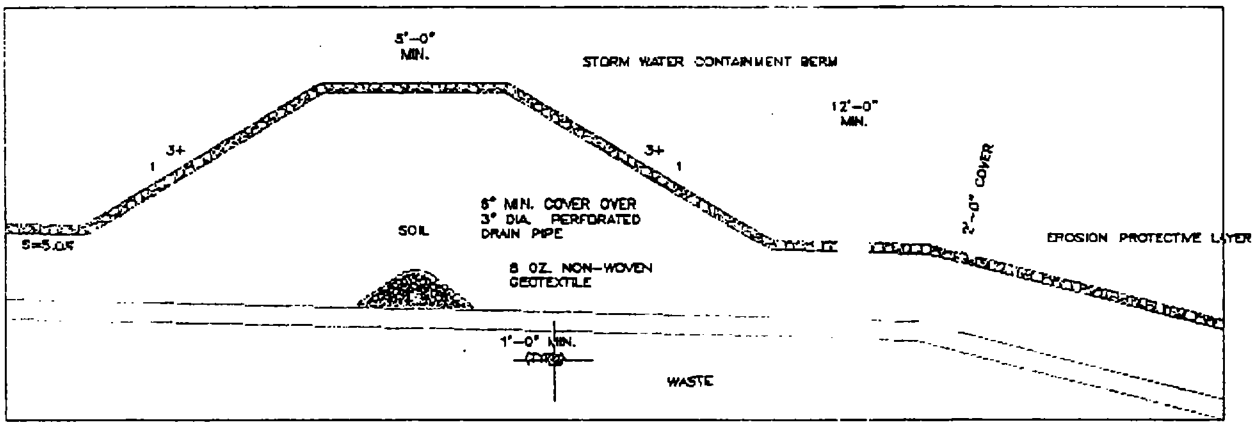
R = Rainfall (precipitation) factor.

K = Soil erodibility factor in tons per acre per year per unit of R.

LS = Topographic factor (length and steepness of slope).

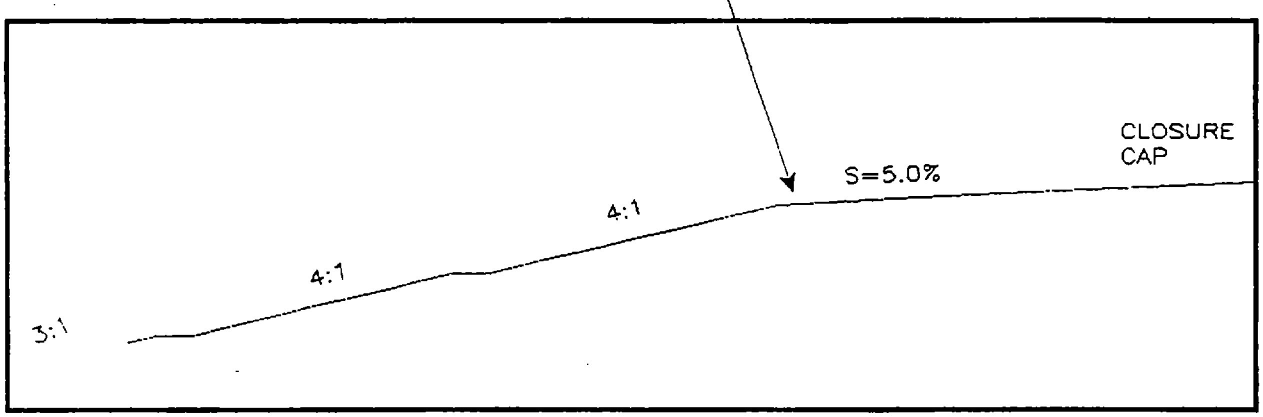
HANSEN ALLEN & LUCE inc ENGINEERS

CLIENT: Wasatch Regional		SHEET 2 OF 9
PROJECT: Landfill Permit		COMPUTED: GLJ
FEATURE: Erosion Protection		CHECKED: KCS
PROJECT NO.: 113.30.100		DATE: December 2004



TOP OF 4:1 SLOPE SECTION 5/4

N.T.S.



Calculated erosion after applying erosion control measures is determined by applying and erosion control factor (VM) to the universal soil loss equation. The erosion control factor is dependant upon the type and extent to which the erosion control measure is used (ie. vegetative - type and density, mulches - type and thickness, chemical - type and application amount, mechanical - compactive effort, smoothness of surface, etc.).

A. The rainfall (precipitation) factor (R) is obtained from mean annual iso-erodent (R) value maps. The R-value for the facility as obtained from the Tooele area map is:

$$R = 5.5$$

Since R = 5.5 is based on an annual recurrence interval, a correction factor is obtained from the figure below for the 100-yr recurrence interval
For the 100-yr recurrence interval:

$$R = 5.5(2.51) = 13.81$$

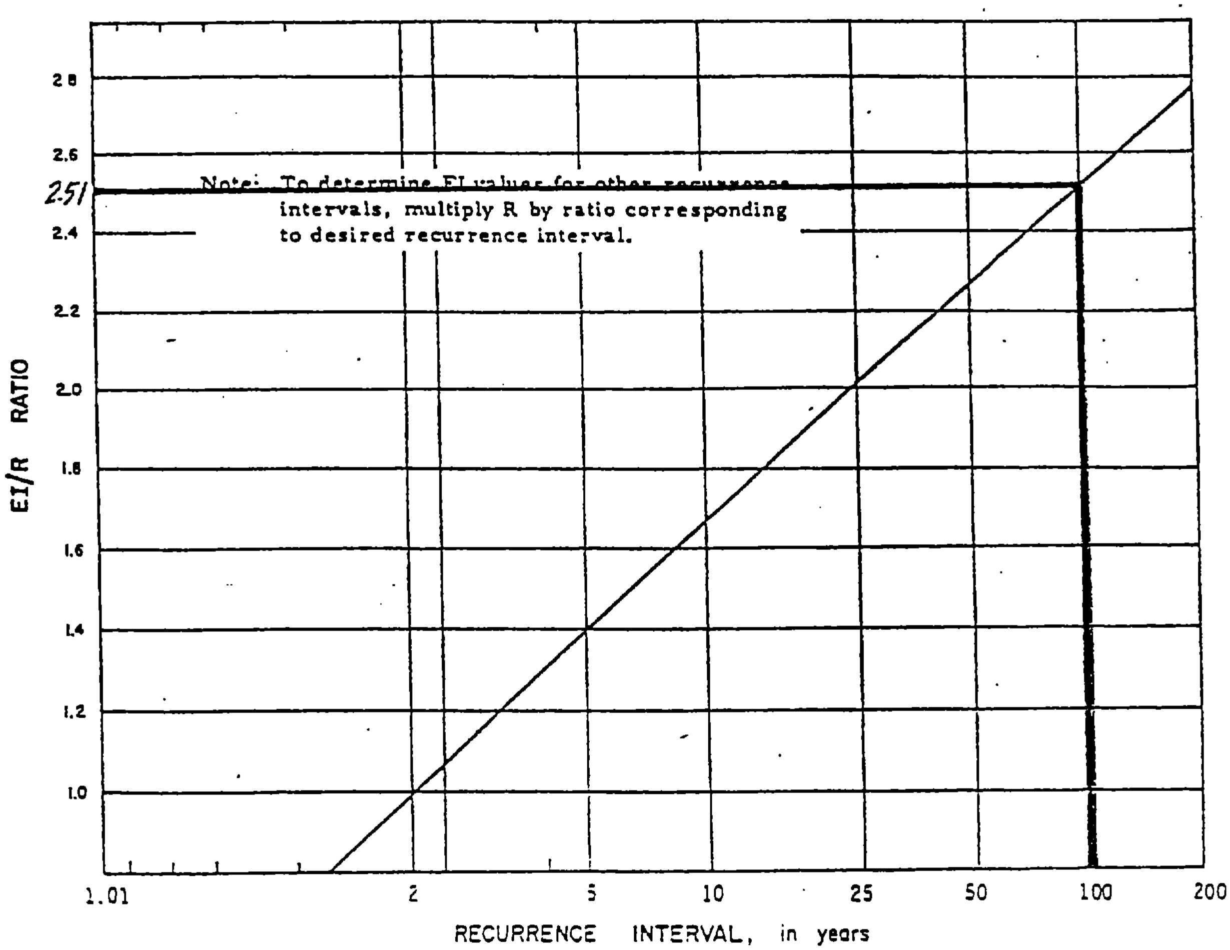


Figure 2-1. The relationship between the EI/R ratio and recurrence interval.

B. Soil erodibility factor (K) is determined using the figures on Sheet 5. The gradation of the materials is based on information from the Kleinfelder soil report.

The worst case condition is represented by the soils whose gradation is on the fine side of the soil gradation envelope. Parameters obtained from the gradation envelope and parameters assumed for use with the nomographs to determine K are:

91 %	silt and very fine sand
9 %	sand
0 %	organic material

Applying the above parameters to the nomographs on Sheet 5 gives a soil erodibility factor (K) equal to 0.75.

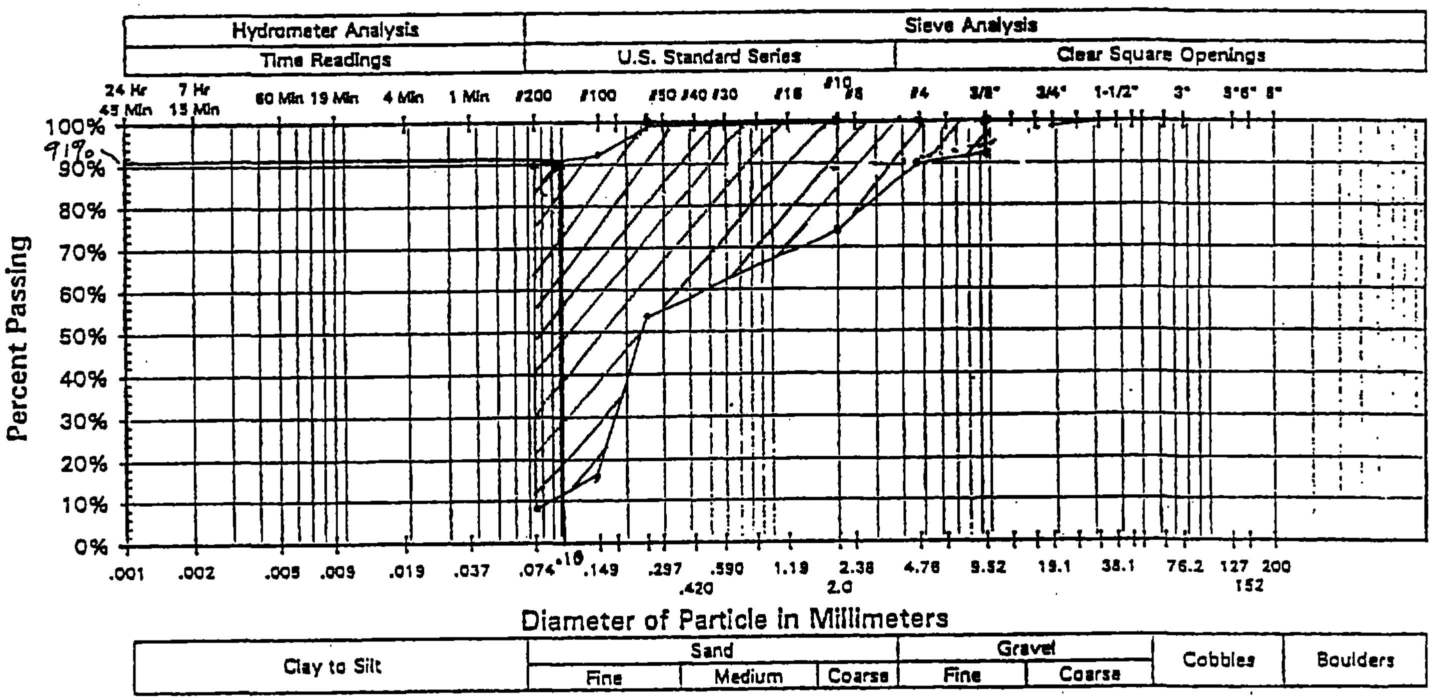
C. The topographic factor (LS) is determined assuming single slopes since runoff will be captured from the 15 percent slope prior to entering the 4H:1V slope by construction of a berm or some form of runoff conveyance channel. The figure on Sheet 2 shows the configuration of the different slope segments that need to be accounted for in the calculations. The LS factor is determined by the following equation:

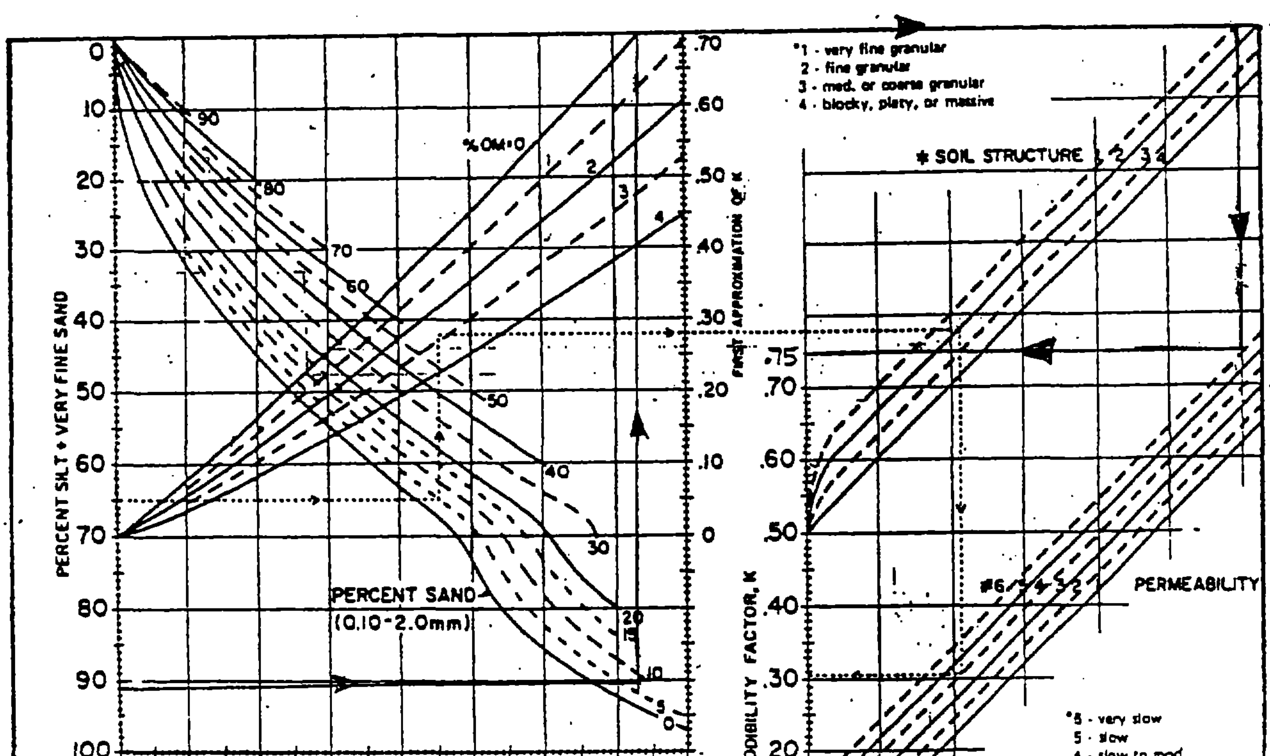
$$LS = \left(\frac{65.41\ s^2}{s^2 + 10,000} + \frac{4.56\ s}{\sqrt{s^2 + 10,000}} + 0.065 \right) \left(\frac{l}{72.6} \right)^m$$

where;

LS	=	topographic factor for slope segment n.
l	=	length of slope segment n.
s	=	slope gradient of segment n in percent.
l	=	slope length
m	=	slope gradient factor

HANSEN ALLEN & LUCE inc ENGINEERS	CLIENT: Wasatch Regional		SHEET 5 OF 9
	PROJECT: Landfill Permit		COMPUTED: GLJ
	FEATURE: Erosion Protection		CHECKED: KCS
	PROJECT NO.: 113.30.100		DATE: December 2004





PROCEDURE: With appropriate data, enter scale at left and proceed to points representing the soil's % sand (0.10–2.0 mm), % organic matter, structure, and permeability, in that sequence. Interpolate between plotted curves. The dotted line illustrates procedure for a soil having: si+vfs 65%, sand 5%, OM 2.8%, structure 2, permeability 4. Solution: K = 0.31.

Figure 2. Nomograph for determining soil erodibility factor K.

The following table provides LS factor values for varying lengths of the 3H:1V, 4H:1V slopes and 5% slopes.

HORIZONTAL DISTANCE ALONG SLOPE (ft)	SLOPE LENGTHS (ft) AND LS FACTOR VALUES					
	33% Slope		4H:1V (25%) Slope		Top of Cap (5%) Slope	
	Slope Length	LS Factor	Slope Length	LS Factor	Slope Length	LS Factor
85	89.51	8.7914				
250			257.69	9.4551		
4100					4105.12	3.4277

A portion of the 5% part of the cap will transition into the 4H:1V slope which will give a resultant LS factor. The formula for combining multiple slopes is:

$$(LS)_n = \frac{(L_{\lambda_n} S_{s_n}) - (L_{\lambda_{n-1}} S_{s_n})\lambda_{n-1}}{ln}$$

$(LS)_n$ = Topographic factor for slope segment n
ln = Length of slope segment n
S_n = Slope gradient in percent of segment n
λ_n = The sum of the slope segment length from the top of the slope to the bottom of slope segment n
S_n = Slope factor for slope segment n
L_n = Length factor for slope segment n

The 5% slope portion would have an LS factor of:

$$(LS)_1 = \frac{(0.53)(100)-(0)(0)}{100} = 0.53$$

The combined 5% into the 4H:1V slope gives an LS factor of:

$$(LS)_2 = \frac{(11.02)(350)-(5.89)(100)}{250} = 13.07$$

D. Potential Erosion Rates without erosion protection where R = 13.81, K = 0.75 and LS as tabulated above are presented in the table below:

POTENTIAL EROSION RATES (A) ASSUMING BARE SOILS

3H:1V (33%) Slope		4H:1V (25%) Slope		5% Top of Cap		5% - Segment 1 of combined slope		5% to 4H:1V - Segment 2 of combined slope	
LS	A (tons/ac/yr)	LS	A (tons/ac/yr)	LS	A (tons/ac/yr)	LS	A (tons/ac/yr)	LS	A (tons/ac/yr)
8.79	91.06	9.46	97.93	3.43	35.50	0.53	5.49	13.07	135.37

E. Required Stone Mulch

The amount of stone mulch required to limit soil loss to one ton per acre per year is determined from the figure on Sheet 9. The figure on Sheet 9 shows the amount of stone mulch required to reduce the erosion potential.

For the 3V:1H (33%) Slope:

Approximately 350 tons per acre of stone mulch is required. The required thickness of stone mulch is:

t = (Required tons/acre of stone mulch x 2000 lbs/ton x 12 in/ft)/
(43560 ft^2/acre x stone mulch density lbs/ft^3)

Assuming a stone mulch density of 110 lbs/ft^3

t = 350(2000)(12)/(43560)(110) = 1.75 in.

For the 4V:1H (25%) Slope:

Approximately 370 tons per acre of stone mulch is required. The required thickness of stone mulch is:

t = 370(2000)(12)/(43560)(110) = 1.85 in.

For the 5% top of cover Slope:

Approximately 150 tons per acre of stone mulch is required. The required thickness of stone mulch is:

t = 150(2000)(12)/(43560)(110) = 0.75 in.

For the 5% - Segment 1 of the Combined Slope:

Approximately 35 tons per acre of stone mulch is required. The required thickness of stone mulch is:

t = 35(2000)(12)/(43560)(110) = 0.18 in.

For the 5% to 4H:1V - Segment 2 of the Combined Slope:

Approximately 525 tons per acre of stone mulch is required. The required thickness of stone mulch is:

t = 525(2000)(12)/(43560)(110) = 2.63 in.

F. Required Vegetative Cover

If a vegetative cover of grass is used instead of the stone mulch, the amount of cover required is determined from the figure on Sheet 9. In order to provide the same prevention as the stone mulch, or 1-ton/acre soil loss at failure, the VM factor required is calculated by the following equation:

VM = 1/A

For the 3V:1H (33%) Slope:

VM = 1/91.06 = 0.01

Percent Ground Cover of Grass = 93% (Regardless of tall weeds)

For the 4V:1H (33%) Slope:

VM = 1/97.93 = 0.01

Percent Ground Cover of Grass = 93% (Regardless of tall weeds)

For the 5% top of cap Slope:

VM = 1/35.5 = 0.03

Percent Ground Cover of Grass = 87% (Regardless of tall weeds)

For the 5% - Segment 1 of the Combined Slope:

VM = 1/5.49 = 0.18

Percent Ground Cover of Grass = 25% (Regardless of tall weeds)

For the 5% to 4H:1V - Segment 2 of the Combined Slope:

VM = 1/135.37 = 0.007

Percent Ground Cover of Grass = 95% (Regardless of tall weeds)

CLIENT:	Wasatch Regional	SHEET 9 OF 9	
PROJECT:	Landfill Permit	COMPUTED: GLJ	
FEATURE:	Erosion Protection	CHECKED: KCS	
PROJECT NO.:	113.30.100	DATE: December 2004	

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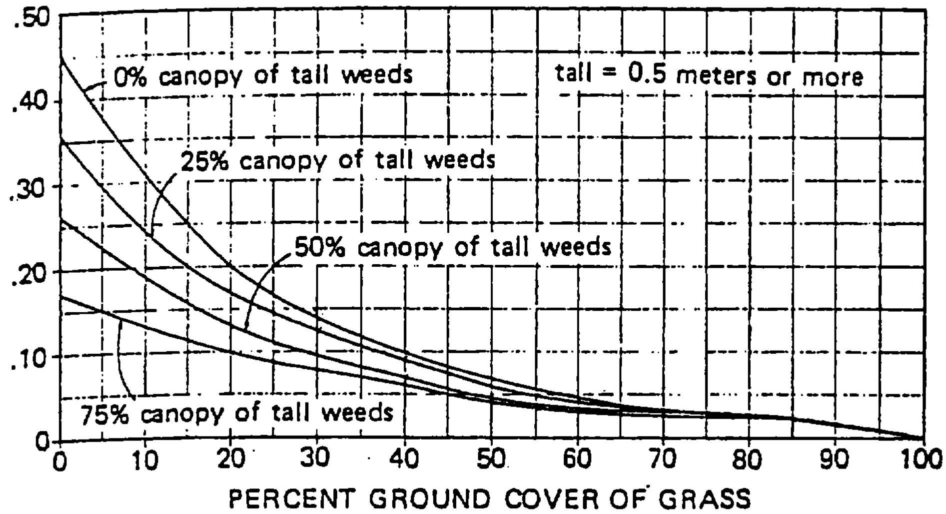

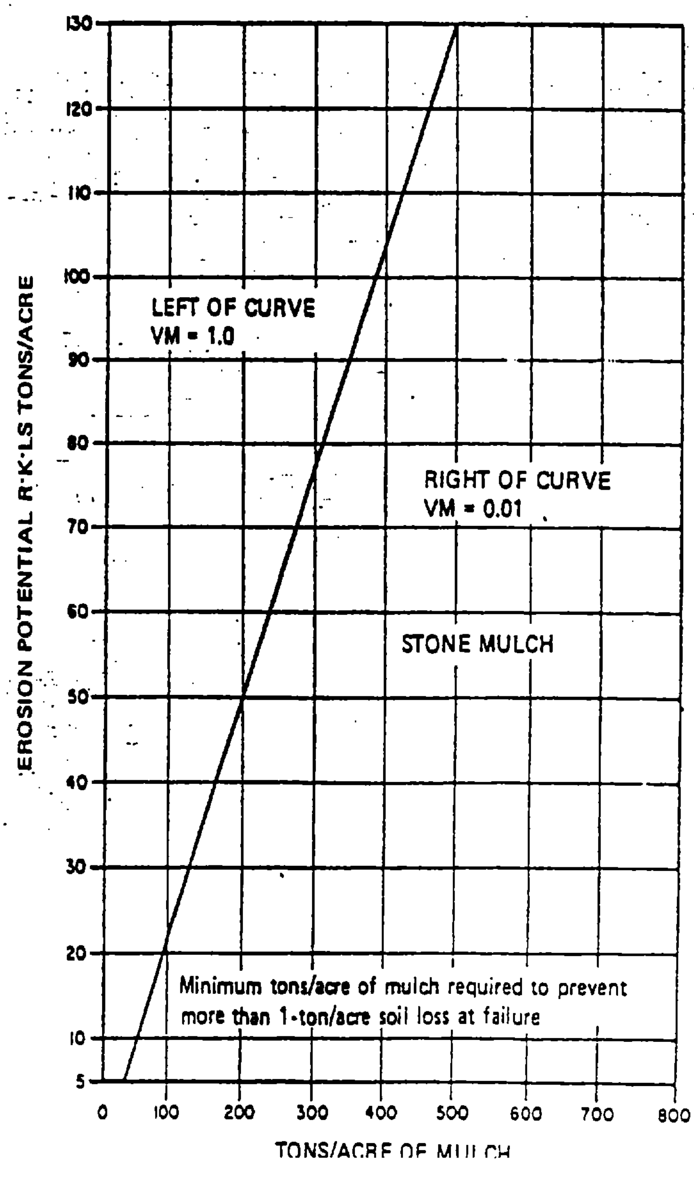

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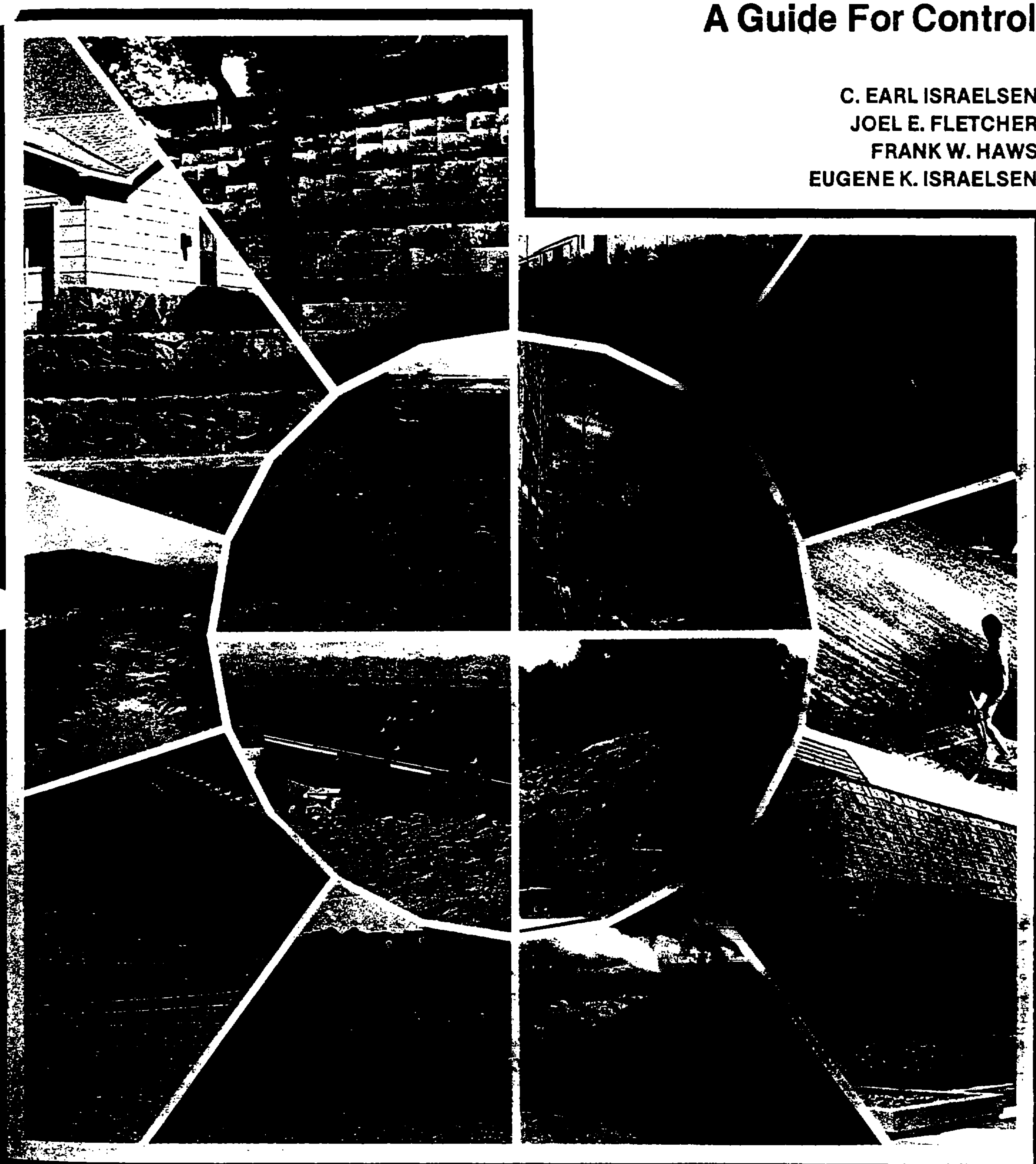

Utah Water Research Laboratory
College of Engineering
Utah State University
Logan, Utah 84322-8200

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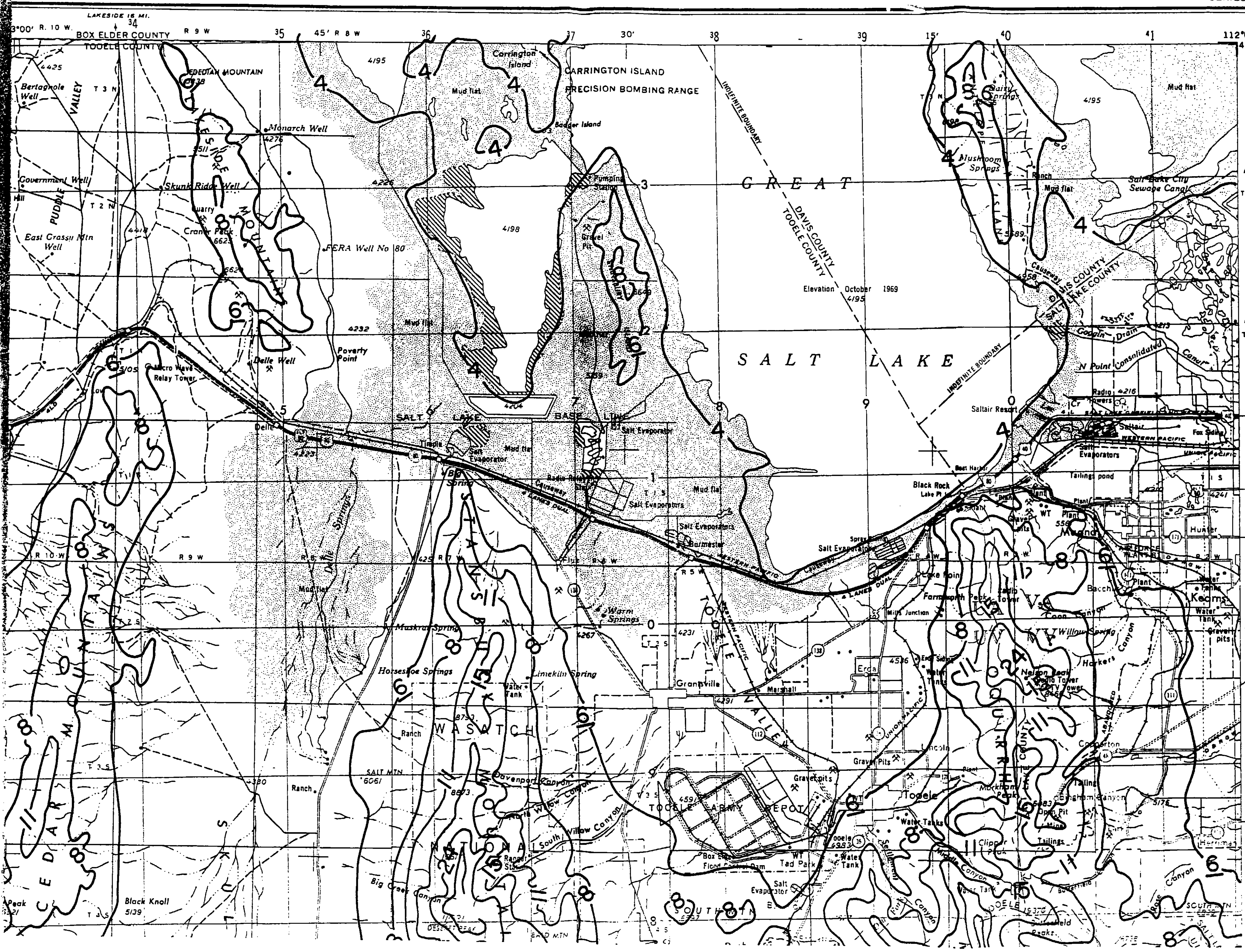
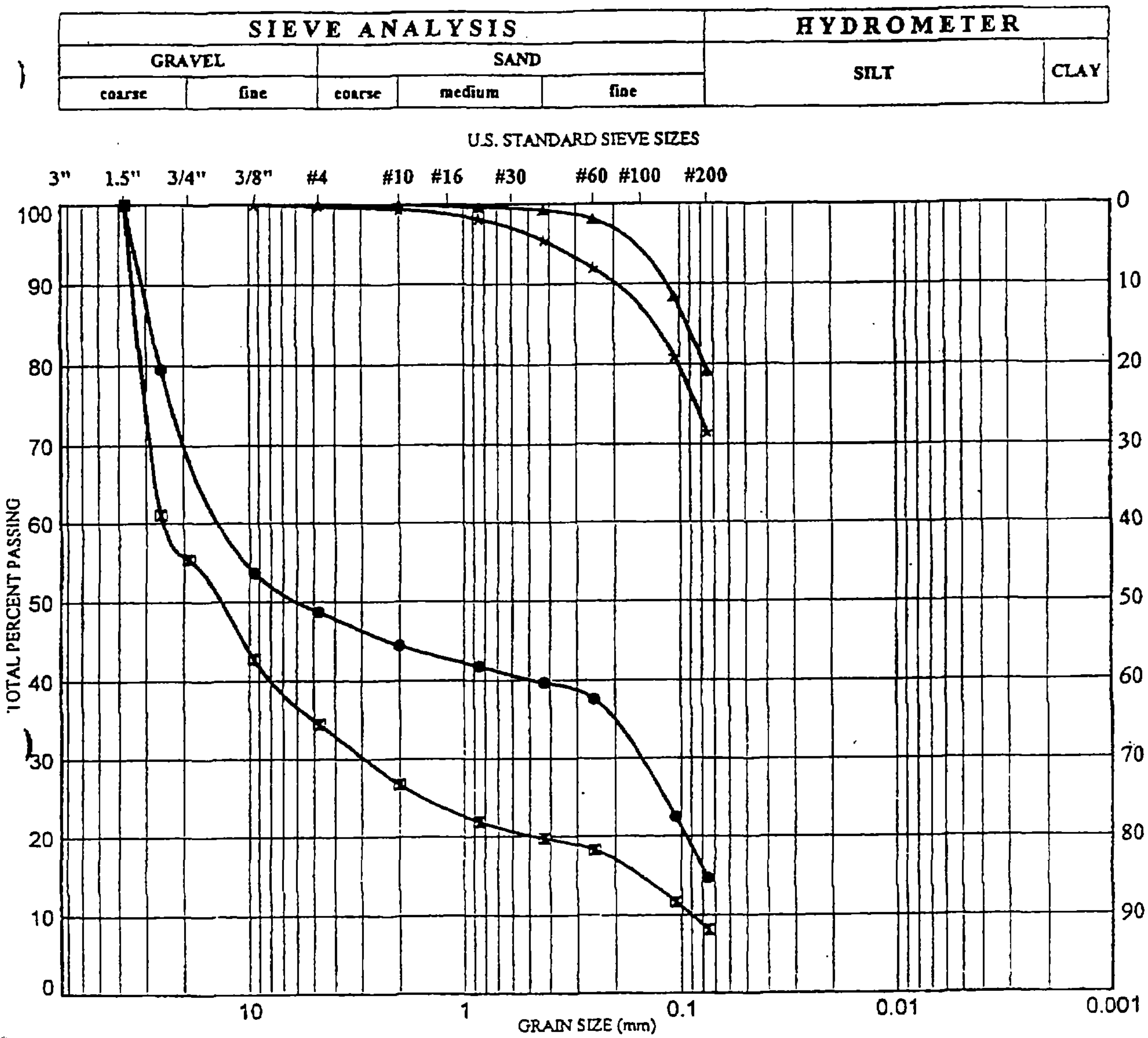




Symbol	Sample	Depth (ft)	USCS Soil Description	USCS Classification
●	TP-8	2.0	Silty GRAVEL with sand	GM
⊠	TP-9	1.0	Poorly Graded GRAVEL with silt and sand	GP-GM
▲	TP-11	1.0	SILTY CLAY with sand	CL-ML
★	TP-15	1.0	Lean CLAY with sand	CL

KLEINFELDER

PROJECT NO. 35467.003

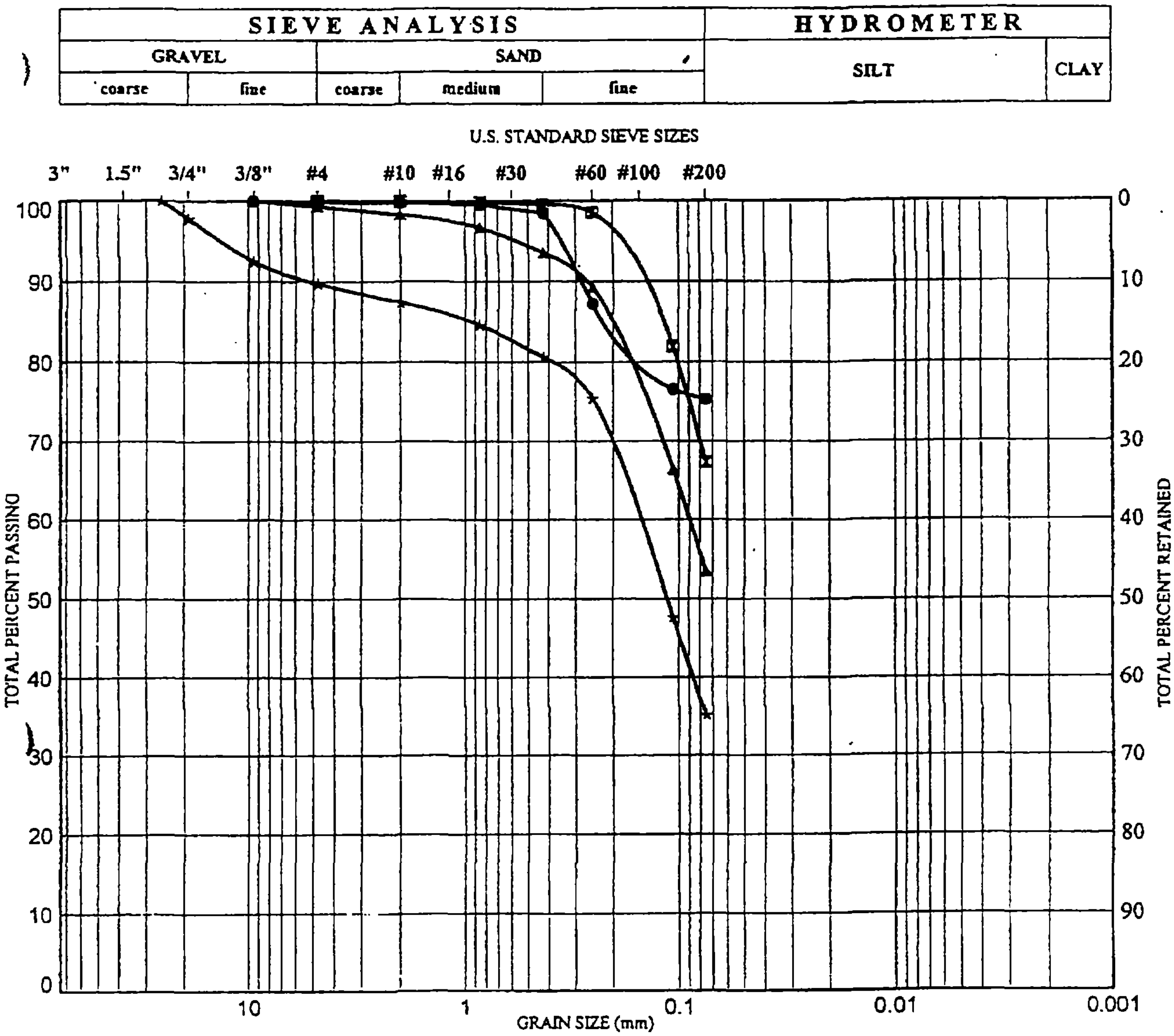
Wasatch Regional Solid Waste Landfill

Tooele County, Utah

GRAIN SIZE DISTRIBUTION

FIGURE

B-18



Symbol	Sample	Depth (ft)	USCS Soil Description	USCS Classification
●	TP- 5	2.0	SILT with sand	ML
⊠	TP- 5	8.0	Sandy SILT	ML
▲	TP- 6	1.0	Sandy SILT	ML
★	TP- 7	1.0	Silty SAND	SM

KLEINFELDER

PROJECT NO. 35467.003

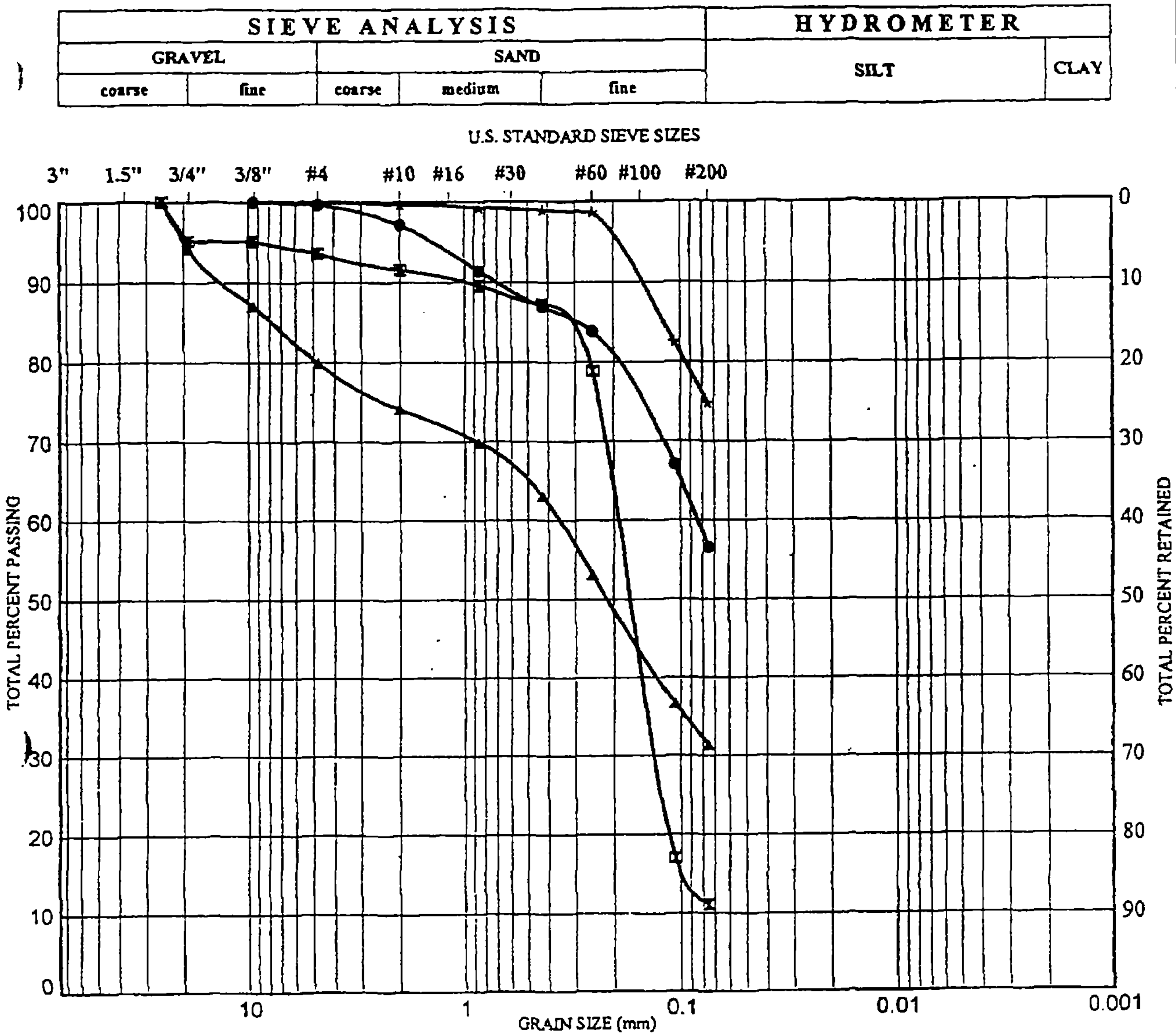
Wasatch Regional Solid Waste Landfill

Tooele County, Utah

GRAIN SIZE DISTRIBUTION

FIGURE

B-17



Symbol	Sample	Depth (ft)	USCS Soil Description	USCS Classification
●	TP- 1	1.0	Sandy Lean CLAY	CL
⊠	TP- 1	10.0	Poorly Graded SAND with silt	SP-SM
▲	TP- 2	11.0	Silty SAND with gravel	SM
★	TP- 3	2.0	SILT with sand	ML

KLEINFELDER

PROJECT NO. 35467.003

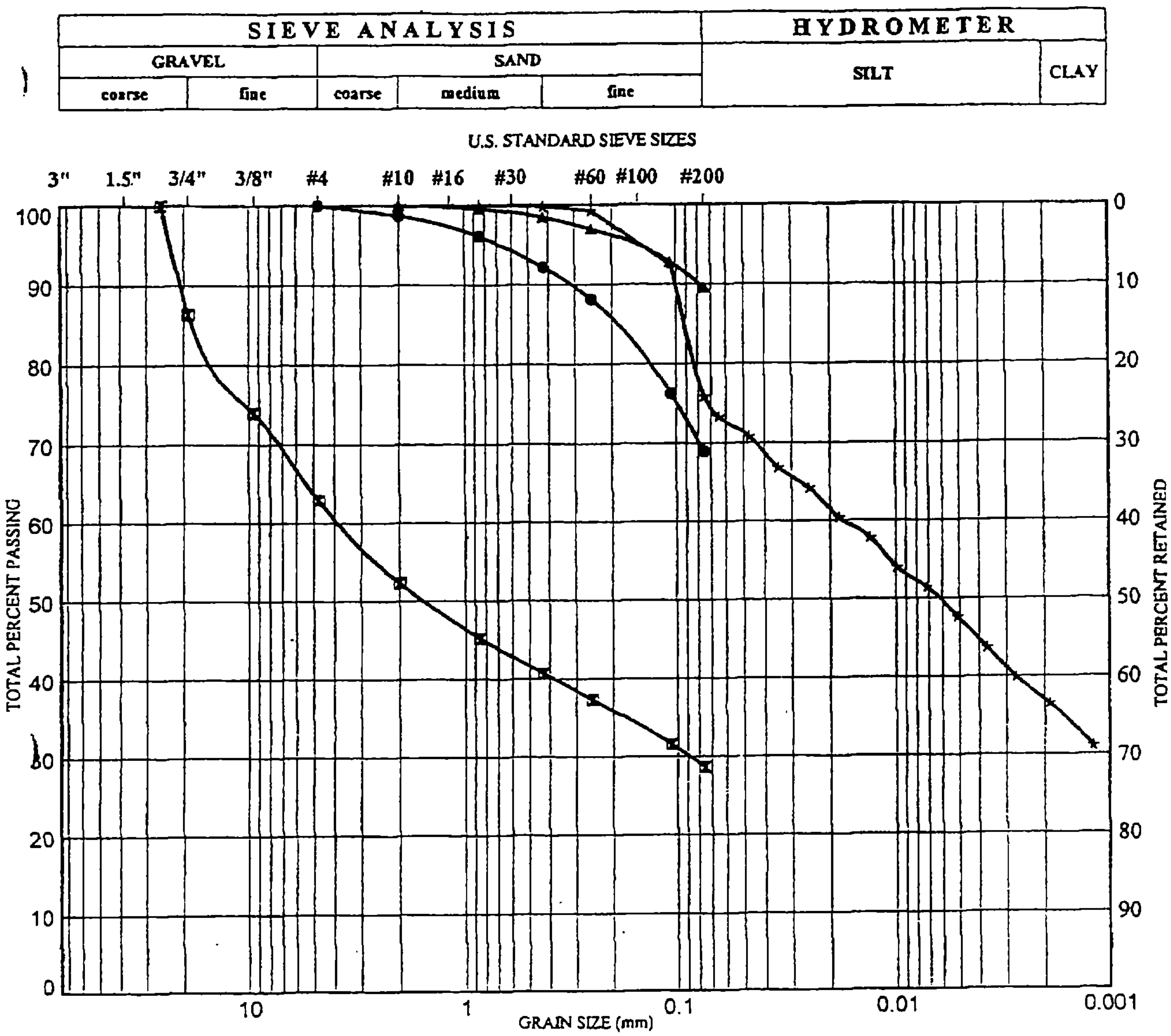
Wasatch Regional Solid Waste Landfill

Tooele County, Utah

GRAIN SIZE DISTRIBUTION

FIGURE

B-16



Symbol	Sample	Depth (ft)	USCS Soil Description	USCS Classification
●	B- 7	5.0	Sandy SILT	ML
⊠	B- 7	25.0	Clayey GRAVEL with sand	GC
▲	B- 9	2.0	Lean CLAY	CL
★	B-10	35.0	Lean CLAY with sand	CL

KLEINFELDER

PROJECT NO. 35467.003

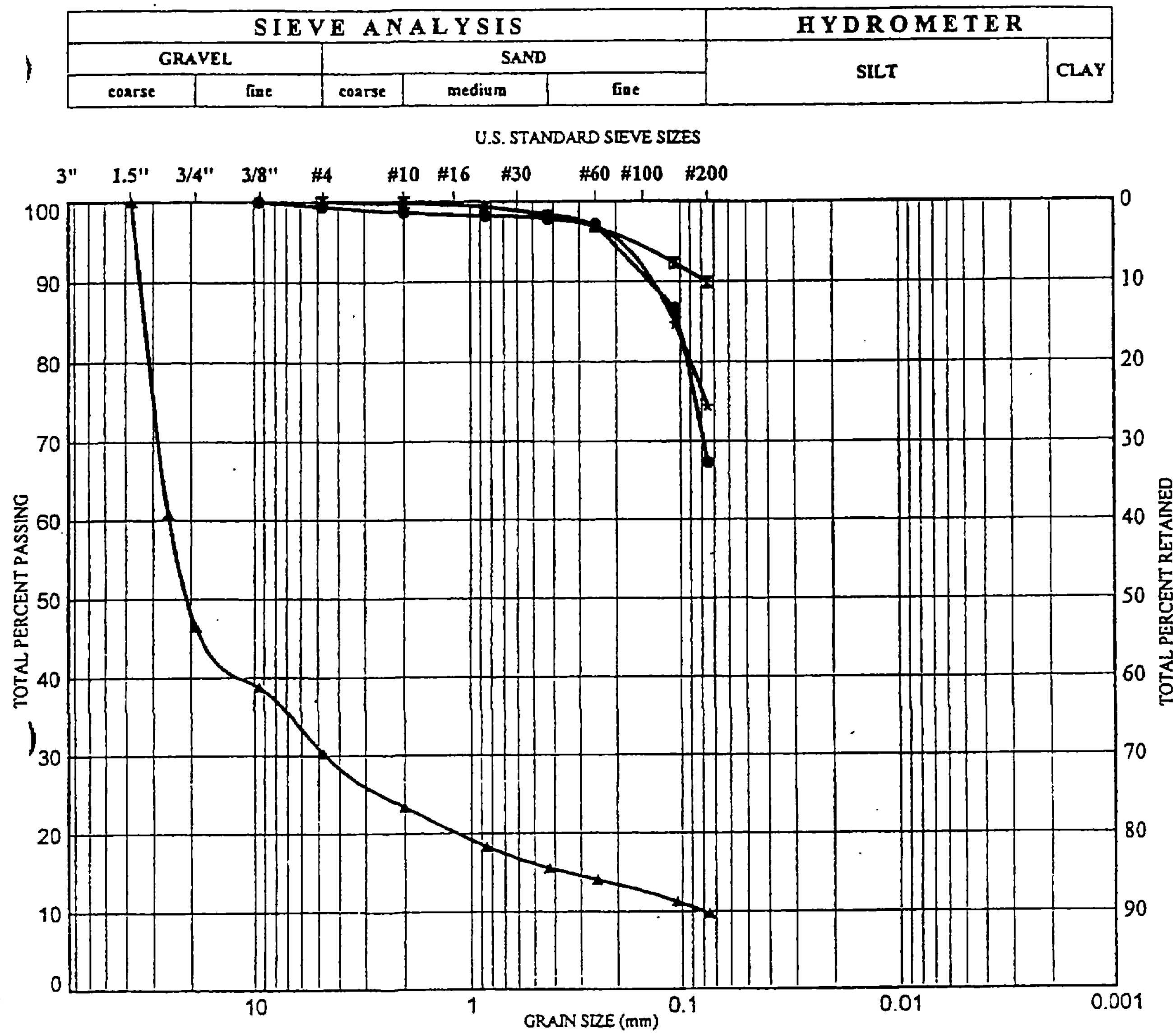
Wasatch Regional Solid Waste Landfill

Tooele County, Utah

GRAIN SIZE DISTRIBUTION

FIGURE

B-15



Symbol	Sample	Depth (ft)	USCS Soil Description	USCS Classification
●	B- 4	45.0	Sandy SILT	ML
⊠	B- 5	25.0	Lean CLAY	CL
▲	B- 5	35.0	Poorly Graded GRAVEL with silt and sand	GP-GM
★	B- 6	2.0	SILT with sand	ML

KLEINFELDER

PROJECT NO. 35467.003

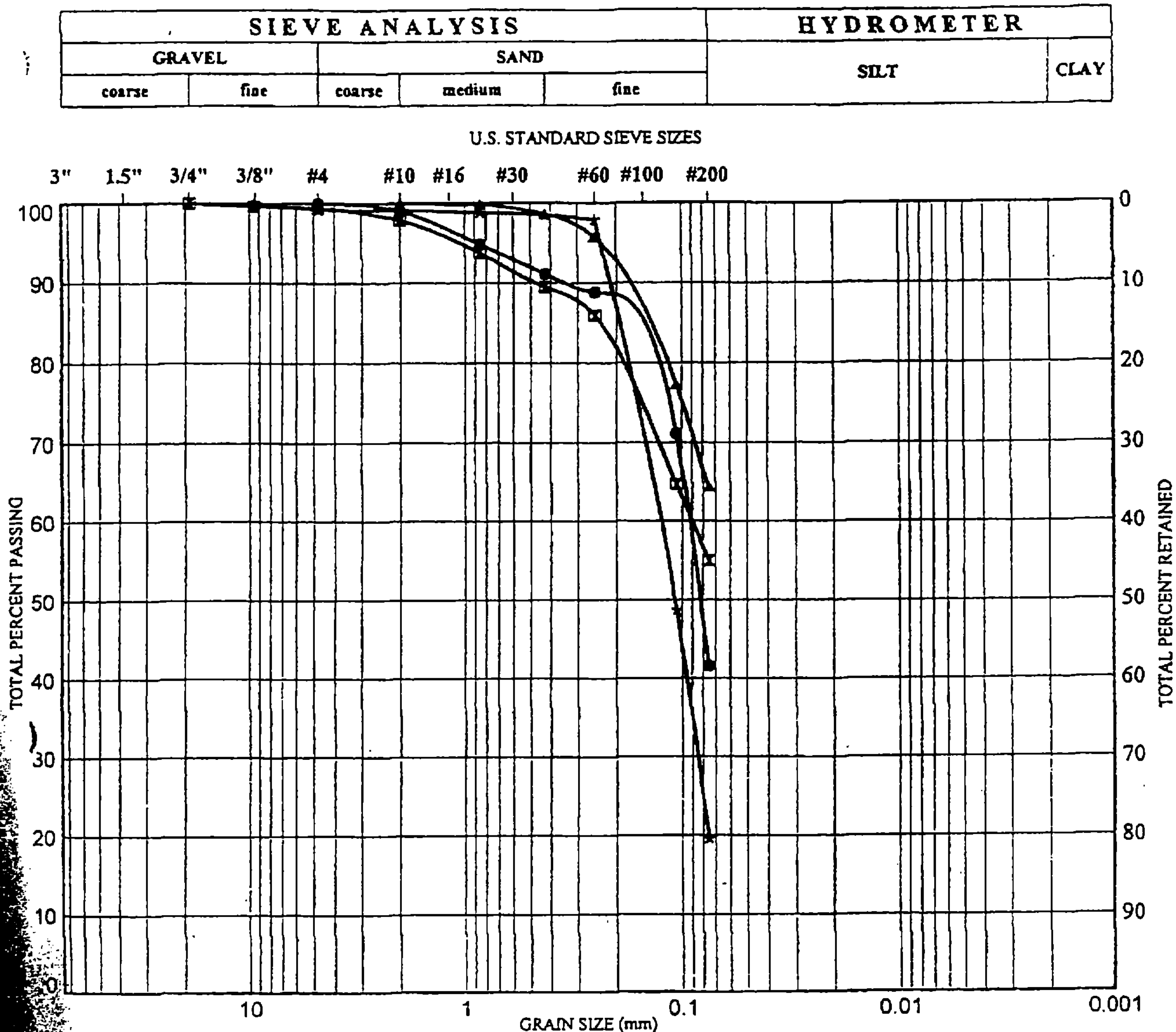
Wasatch Regional Solid Waste Landfill

Tooele County, Utah

GRAIN SIZE DISTRIBUTION

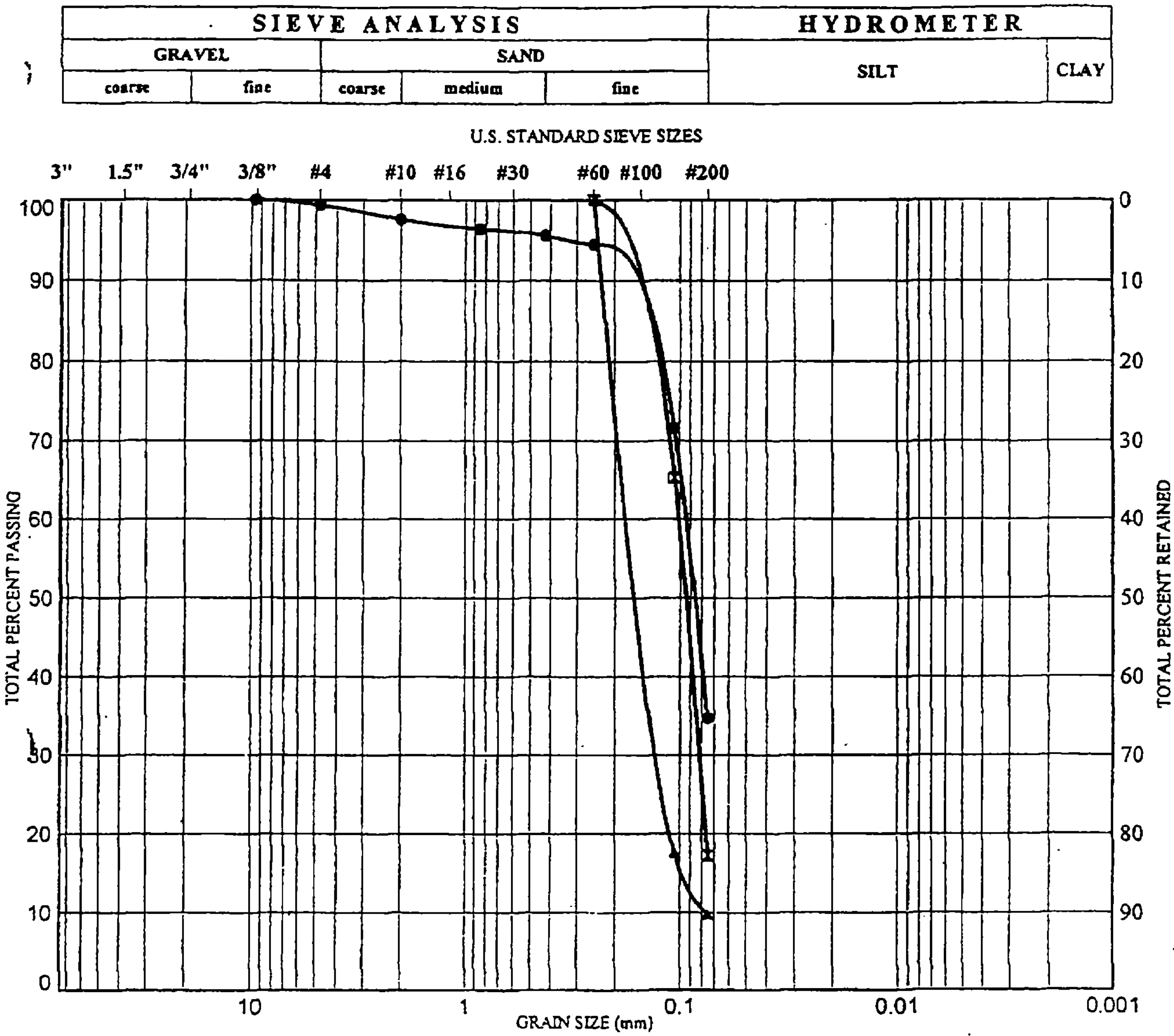
FIGURE

B-14



Symbol	Sample	Depth (ft)	USCS Soil Description	USCS Classification
●	B- 1	25.0	Silty SAND	SM
⊠	B- 2	2.0	Sandy Lean CLAY	CL
▲	B- 4	2.0	Sandy SILTY CLAY	CL-ML
	B- 4	30.0	Silty SAND	SM

KLEINFELDER	Wasatch Regional Solid Waste Landfill Tooele County, Utah	*FIGURE*
NO. 35467.003	**GRAIN SIZE DISTRIBUTION**	**B-13**



Symbol	Sample	Depth (ft)	USCS Soil Description	USCS Classification
●	B- 1(i)	15.0	Silty SAND	SM
⧗	B- 2(i)	20.0	Silty SAND	SM
▲	B- 3(i)	10.0	SAND - w/some silt	SP-SM

KLEINFELDER

PROJECT NO. 31168.001

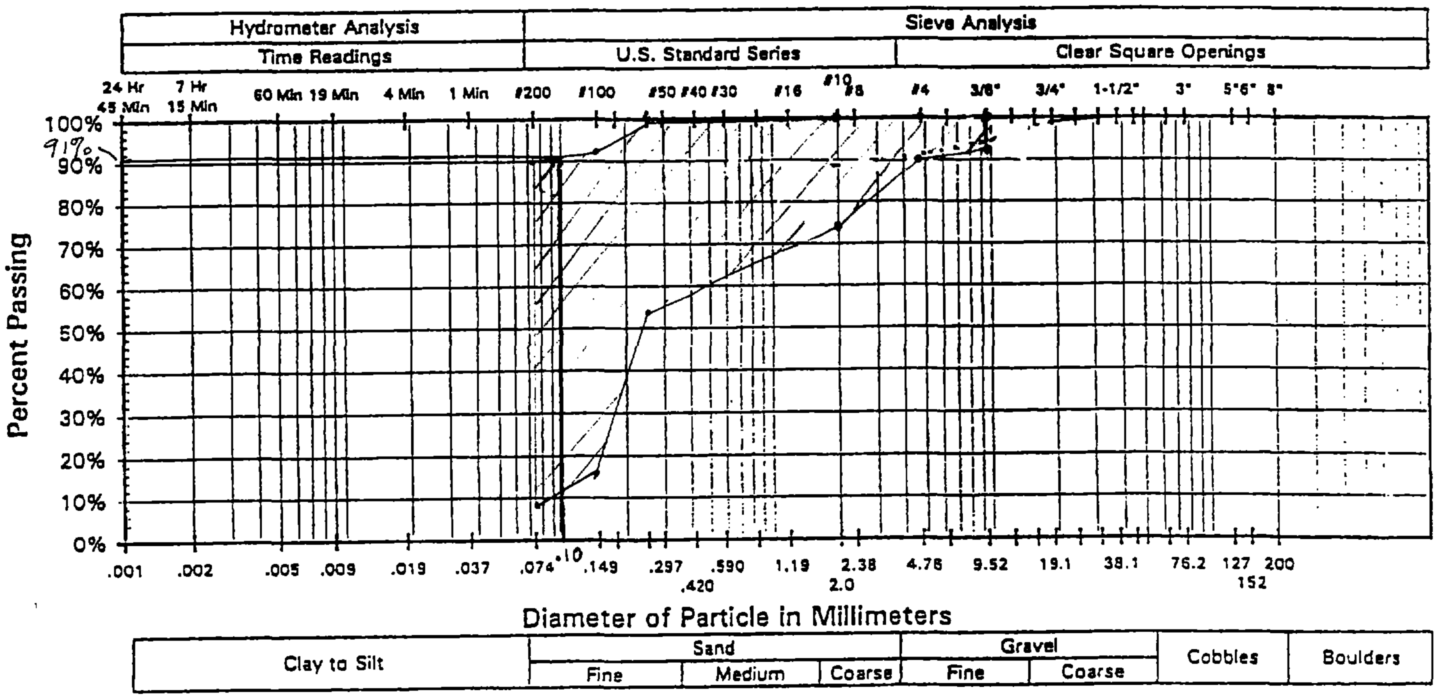
Wasatch Regional Solid Waste Landfill

Tooele County, Utah

GRAIN SIZE DISTRIBUTION

FIGURE

B-12



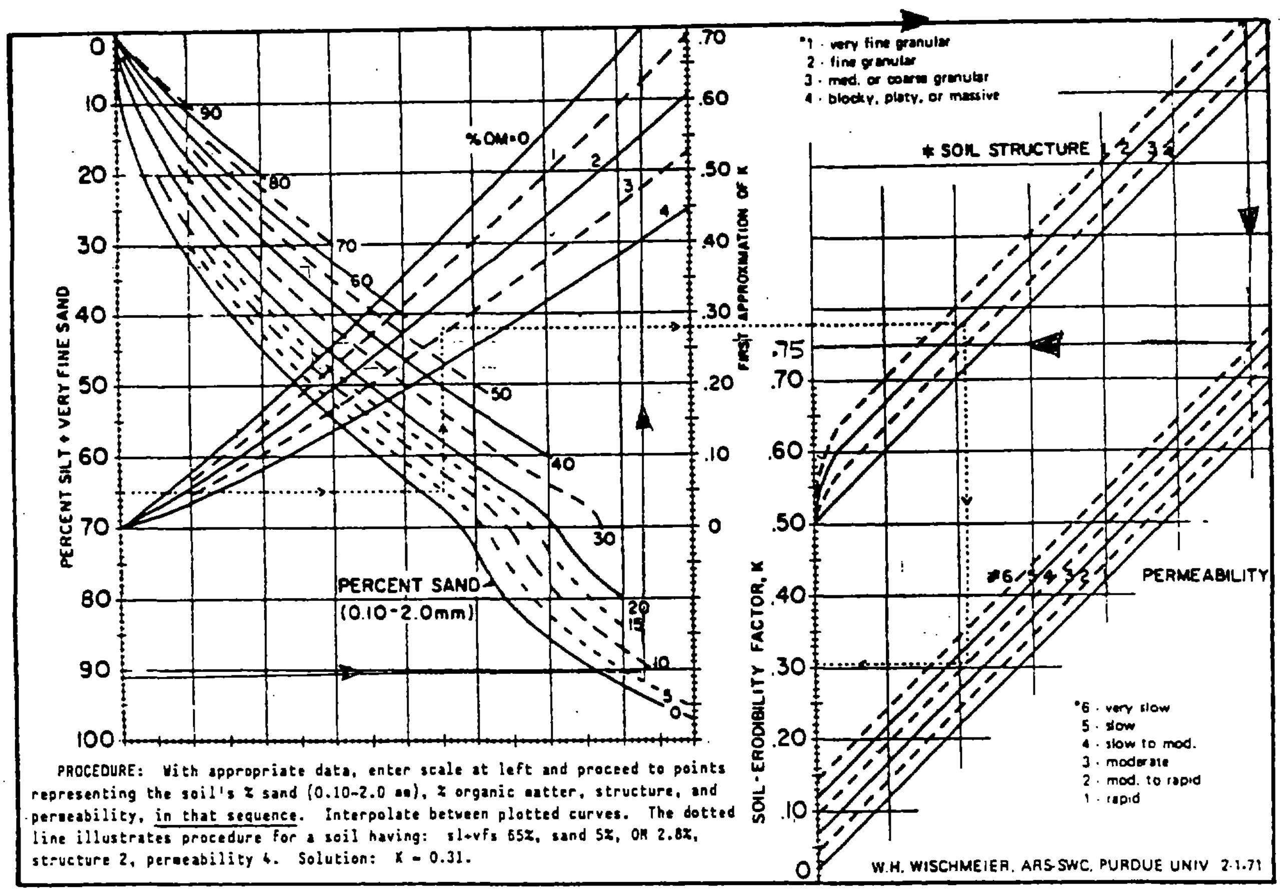
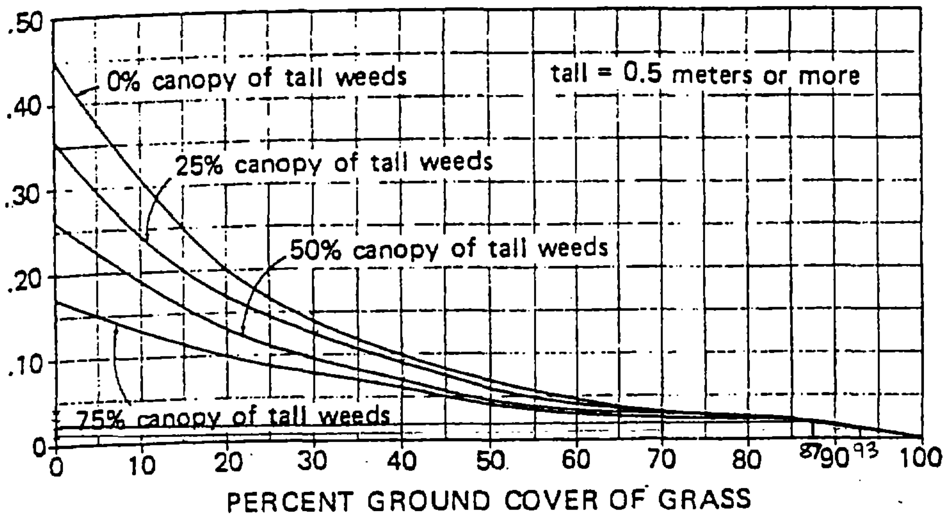
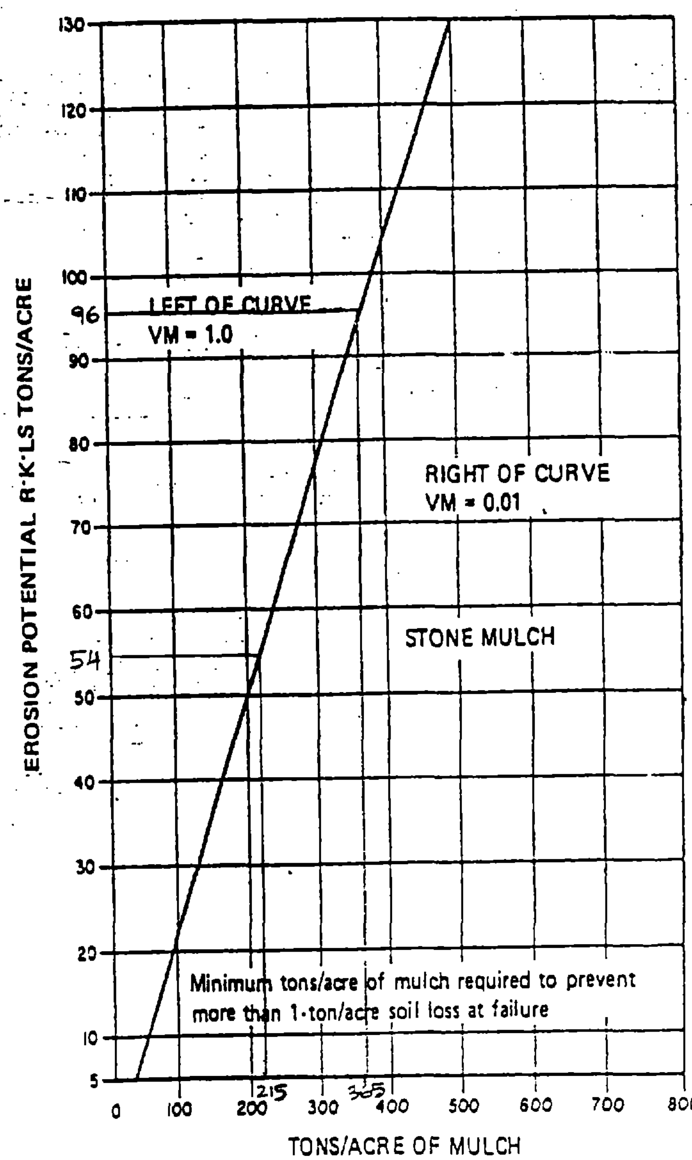


Figure 2. Nomograph for determining soil erodibility factor K.

HANSEN ALLEN & LUCE INC ENGINEERS	*CLIENT:* Allied Waste		*SHEET* 8 *OF* 8
	PROJECT: Wasatch Regional		*COMPUTED:* GLJ
	FEATURE: Erosion Protection		*CHECKED:*
	PROJ. NO.: 113.30.100		*DATE:* September 2004





I. Purpose and Procedure.

The purpose of these calculations is determine which erosion protection measure to use and how to apply it. The closure cap will consist of a 4H:1V slope extending up from the top of the cell embankments. The embankments will consist of a 3H:1V slope from the top of the embankment down to the ground surface. The top of the closure cap will have a 5% slope. There will be a 5% section between the berm on the closure top that will combine with the 4H:1V slope.

The procedure used to determine the allowable slope lengths between the bench areas of the closure cap slopes is taken from the publication "Erosion and Sedimentation in Utah - A Guide for Control", Utah Water Research Laboratory, February 1984. This publication is specific to Utah. The figure presented on Sheet 2 presents a cross-section showing the configuration of the area contributing runoff to the slopes of the closure cap. Each slope between bench areas will consist of relatively uniform lengths such that the calculations for one slope length will be representative for each slope segment between benches along the slopes of the closure cap.

II. The procedure from the above publication uses the Universal Soil Loss Equation (in modified form to represent Utah's climatic and topographic conditions) to estimate the soil erosion potential of the surface soils assuming no application of erosion control measures. Erosion control measures to be implemented are based on the soil erosion potential calculated.

The universal soil loss equation used to calculate soil erosion potential is:

$$A = R \cdot K \cdot LS$$

where;

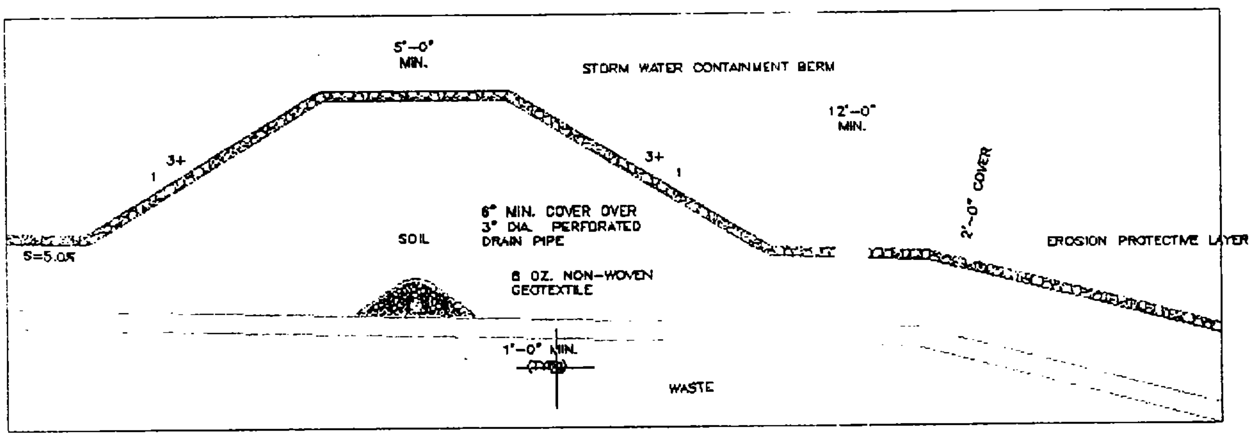
A = Computed amount of soil loss per unit area for the time interval represented by factor R, generally in tons per acre per year.

R = Rainfall (precipitation) factor.

K = Soil erodibility factor in tons per acre per year per unit of R.

LS = Topographic factor (length and steepness of slope).

HANSEN ALLEN & LUCE inc ENGINEERS	CLIENT: Wasatch Regional PROJECT: Landfill Permit FEATURE: Erosion Protection PROJECT NO.: 113.30.100	SHEET 2 OF 9 COMPUTED: GLJ CHECKED: KCS DATE: December 2004



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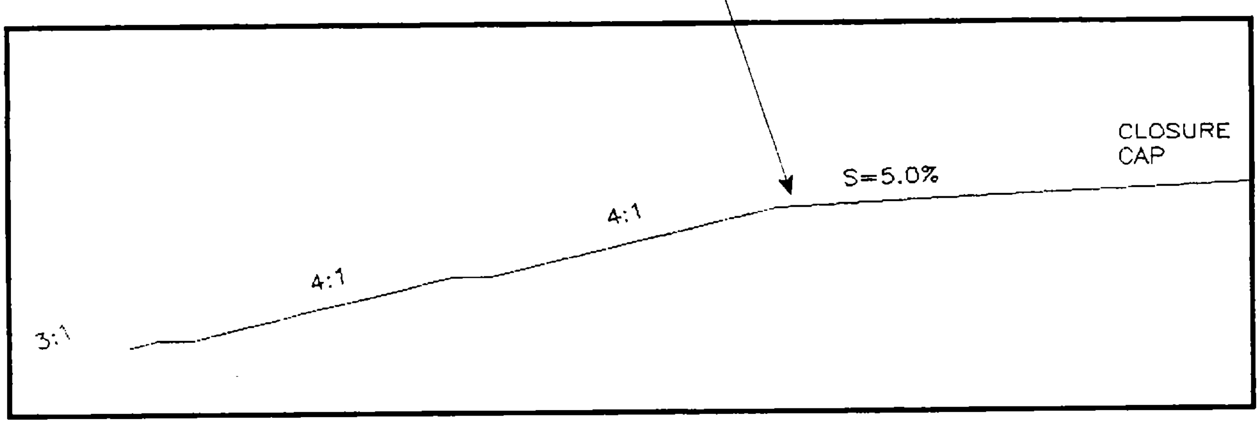


Calculated erosion after applying erosion control measures is determined by applying and erosion control factor (VM) to the universal soil loss equation. The erosion control factor is dependant upon the type and extent to which the erosion control measure is used (ie. vegetative - type and density, mulches - type and thickness, chemical - type and application amount, mechanical - compactive effort, smoothness of surface, etc.).

A. The rainfall (precipitation) factor (R) is obtained from mean annual iso-erodent (R) value maps. The R-value for the facility as obtained from the Tooele area map is:

$$R = 5.5$$

Since R = 5.5 is based on an annual recurrence interval, a correction factor is obtained from the figure below for the 100-yr recurrence interval
For the 100-yr recurrence interval:

$$R = 5.5(2.51) = 13.81$$

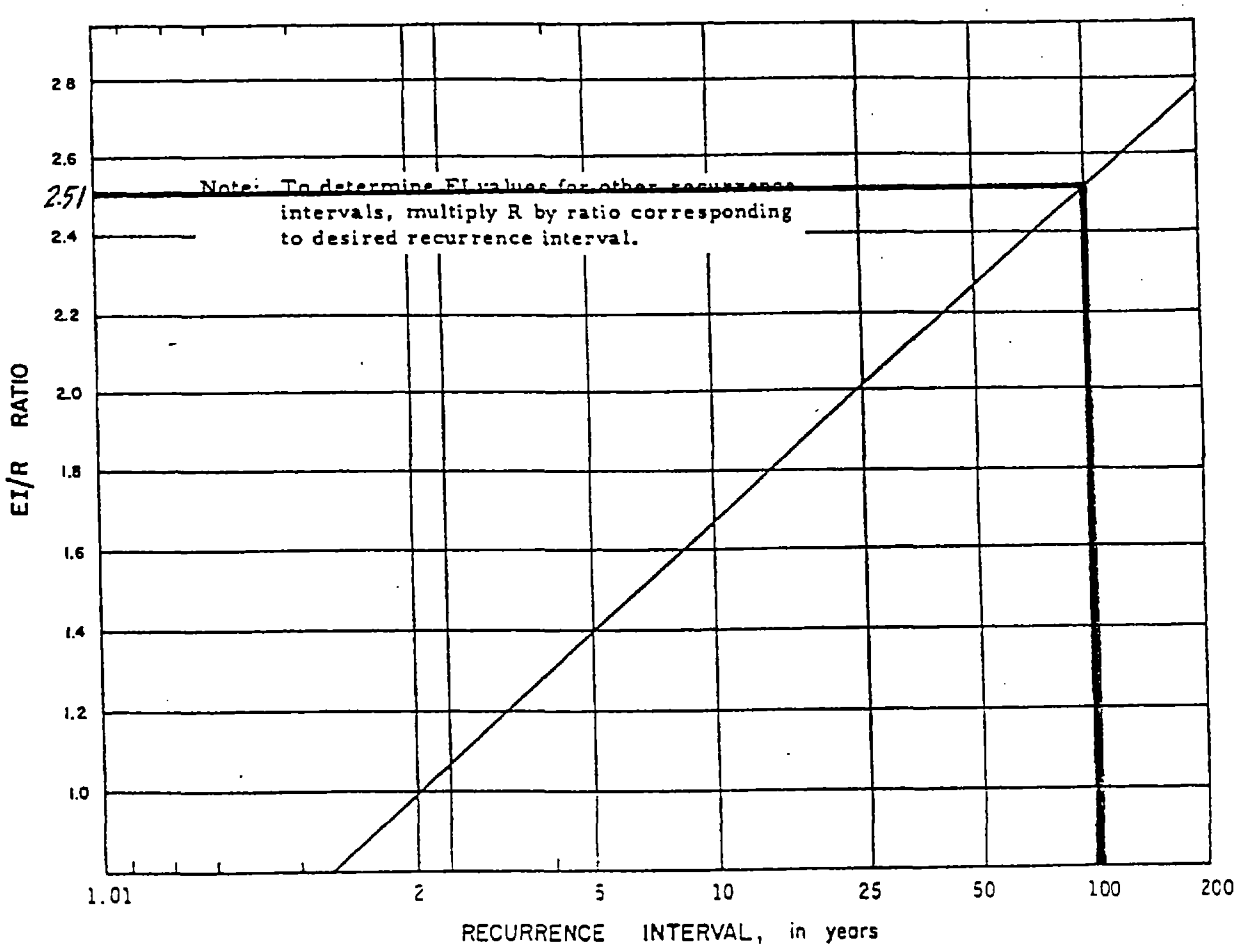


Figure 2-1. The relationship between the EI/R ratio and recurrence interval.

B. Soil erodibility factor (K) is determined using the figures on Sheet 5. The gradation of the materials is based on information from the Kleinfelder soil report.

The worst case condition is represented by the soils whose gradation is on the fine side of the soil gradation envelope. Parameters obtained from the gradation envelope and parameters assumed for use with the nomographs to determine K are:

91 %	silt and very fine sand
9 %	sand
0 %	organic material

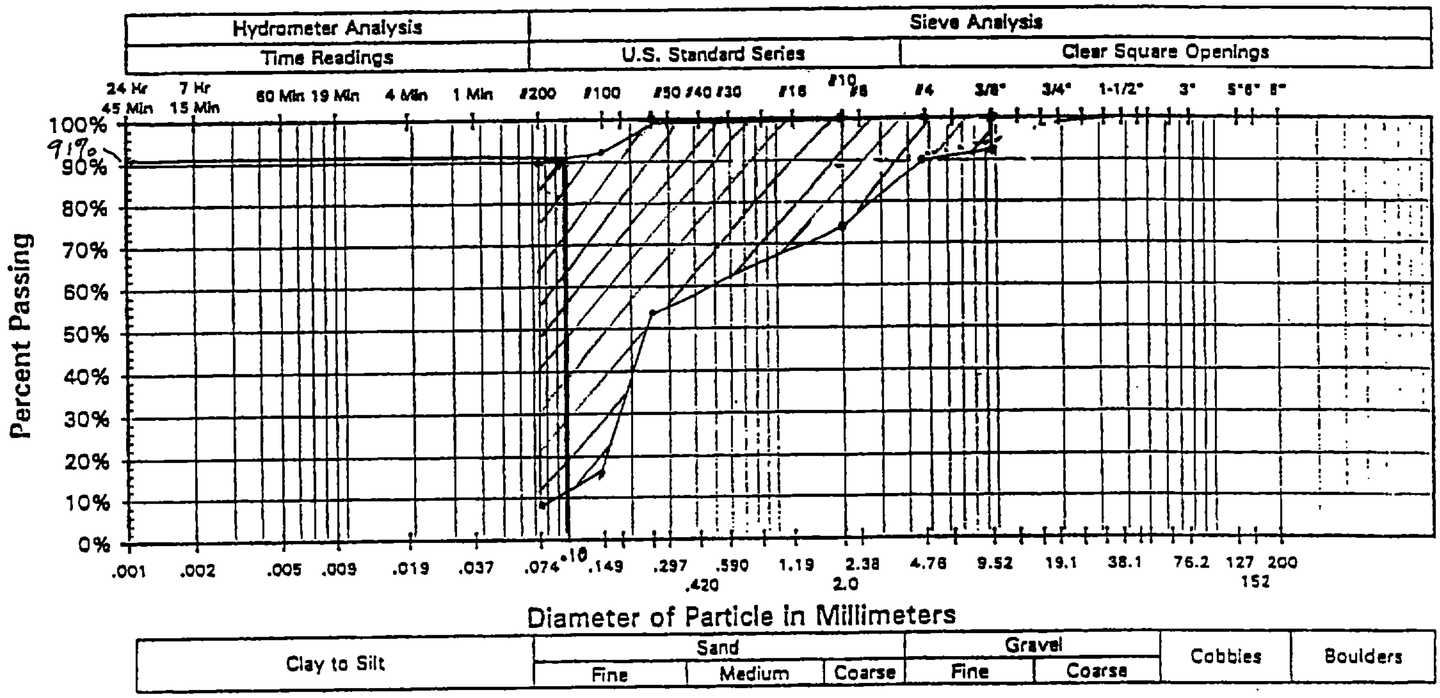
Applying the above parameters to the nomographs on Sheet 5 gives a soil erodibility factor (K) equal to 0.75.

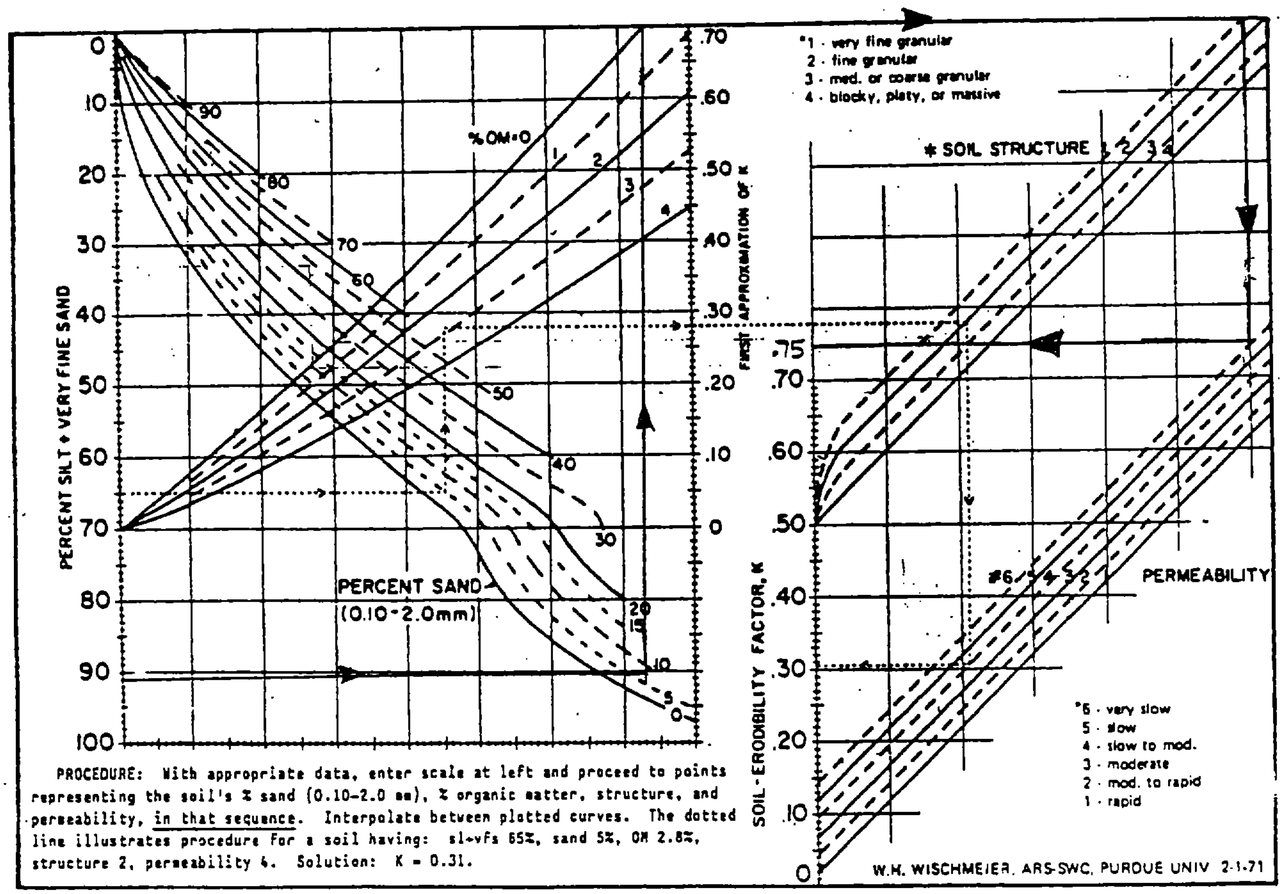
C. The topographic factor (LS) is determined assuming single slopes since runoff will be captured from the 15 percent slope prior to entering the 4H:1V slope by construction of a berm or some form of runoff conveyance channel. The figure on Sheet 2 shows the configuration of the different slope segments that need to be accounted for in the calculations. The LS factor is determined by the following equation:

$$LS = \left(\frac{65.41\, s^2}{s^2 + 10{,}000} + \frac{4.56\, s}{\sqrt{s^2 + 10{,}000}} + 0.065 \right) \left(\frac{l}{72.6} \right)^m$$

where;

LS = topographic factor for slope segment n.
l = length of slope segment n.
s = slope gradient of segment n in percent.
l = slope length
m = slope gradient factor





PROCEDURE: With appropriate data, enter scale at left and proceed to points representing the soil's % sand (0.10-2.0 mm), % organic matter, structure, and permeability, in that sequence. Interpolate between plotted curves. The dotted line illustrates procedure for a soil having: si+vfs 65%, sand 5%, OM 2.8%, structure 2, permeability 4. Solution: K = 0.31.

Figure 2. Nomograph for determining soil erodibility factor K.

The following table provides LS factor values for varying lengths of the 3H:1V, 4H:1V slopes and 5% slopes.

HORIZONTAL DISTANCE ALONG SLOPE (ft)	SLOPE LENGTHS (ft) AND LS FACTOR VALUES					
	33% Slope		4H:1V (25%) Slope		Top of Cap (5%) Slope	
	Slope Length	LS Factor	Slope Length	LS Factor	Slope Length	LS Factor
85	89.51	8.7914				
250			257.69	9.4551		
4100					4105.12	3.4277

A portion of the 5% part of the cap will transition into the 4H:1V slope which will give a resultant LS factor. The formula for combining multiple slopes is:

$$(LS)_n = \frac{(L_{\lambda_n} S_{s_n}) - (L_{\lambda_{n-1}} S_{s_n})\lambda_{n-1}}{\ln}$$

$(LS)_n$ = Topographic factor for slope segment n
ln = Length of slope segment n
S_n = Slope gradient in percent of segment n
λ_n = The sum of the slope segment length from the top of the slope to the bottom of slope segment n
S_n = Slope factor for slope segment n
L_n = Length factor for slope segment n

The 5% slope portion would have an LS factor of:

$$(LS)_1 = \frac{(0.53)(100) - (0)(0)}{100} = 0.53$$

The combined 5% into the 4H:1V slope gives an LS factor of:

$$(LS)_2 = \frac{(11.02)(350) - (5.89)(100)}{250} = 13.07$$

D. Potential Erosion Rates without erosion protection where R = 13.81, K = 0.75 and LS as tabulated above are presented in the table below:

POTENTIAL EROSION RATES (A) ASSUMING BARE SOILS

3H:1V (33%) Slope		4H:1V (25%) Slope		5% Top of Cap		5% - Segment 1 of combined slope		5% to 4H:1V - Segment 2 of combined slope	
LS	A (tons/ac/yr)	LS	A (tons/ac/yr)	LS	A (tons/ac/yr)	LS	A (tons/ac/yr)	LS	A (tons/ac/yr)
8.79	91.06	9.46	97.93	3.43	35.50	0.53	5.49	13.07	135.37

E. Required Stone Mulch

The amount of stone mulch required to limit soil loss to one ton per acre per year is determined from the figure on Sheet 9. The figure on Sheet 9 shows the amount of stone mulch required to reduce the erosion potential.

For the 3V:1H (33%) Slope:

Approximately 350 tons per acre of stone mulch is required. The required thickness of stone mulch is:

t = (Required tons/acre of stone mulch x 2000 lbs/ton x 12 in/ft)/ (43560 ft^2/acre x stone mulch density lbs/ft^3)

Assuming a stone mulch density of 110 lbs/ft^3

t = 350(2000)(12)/(43560)(110) = 1.75 in.

For the 4V:1H (25%) Slope:

Approximately 370 tons per acre of stone mulch is required. The required thickness of stone mulch is:

t = 370(2000)(12)/(43560)(110) = 1.85 in.

For the 5% top of cover Slope:

Approximately 150 tons per acre of stone mulch is required. The required thickness of stone mulch is:

t = 150(2000)(12)/(43560)(110) = 0.75 in.

For the 5% - Segment 1 of the Combined Slope:

Approximately 35 tons per acre of stone mulch is required. The required thickness of stone mulch is:

t = 35(2000)(12)/(43560)(110) = 0.18 in.

For the 5% to 4H:1V - Segment 2 of the Combined Slope:

Approximately 525 tons per acre of stone mulch is required. The required thickness of stone mulch is:

t = 525(2000)(12)/(43560)(110) = 2.63 in.

F. Required Vegetative Cover

If a vegetative cover of grass is used instead of the stone mulch, the amount of cover required is determined from the figure on Sheet 9. In order to provide the same prevention as the stone mulch, or 1-ton/acre soil loss at failure, the VM factor required is calculated by the following equation:

VM = 1/A

For the 3V:1H (33%) Slope:

VM = 1/91.06 = 0.01

Percent Ground Cover of Grass = 93% (Regardless of tall weeds)

For the 4V:1H (33%) Slope:

VM = 1/97.93 = 0.01

Percent Ground Cover of Grass = 93% (Regardless of tall weeds)

For the 5% top of cap Slope:

VM = 1/35.5 = 0.03

Percent Ground Cover of Grass = 87% (Regardless of tall weeds)

For the 5% - Segment 1 of the Combined Slope:

VM = 1/5.49 = 0.18

Percent Ground Cover of Grass = 25% (Regardless of tall weeds)

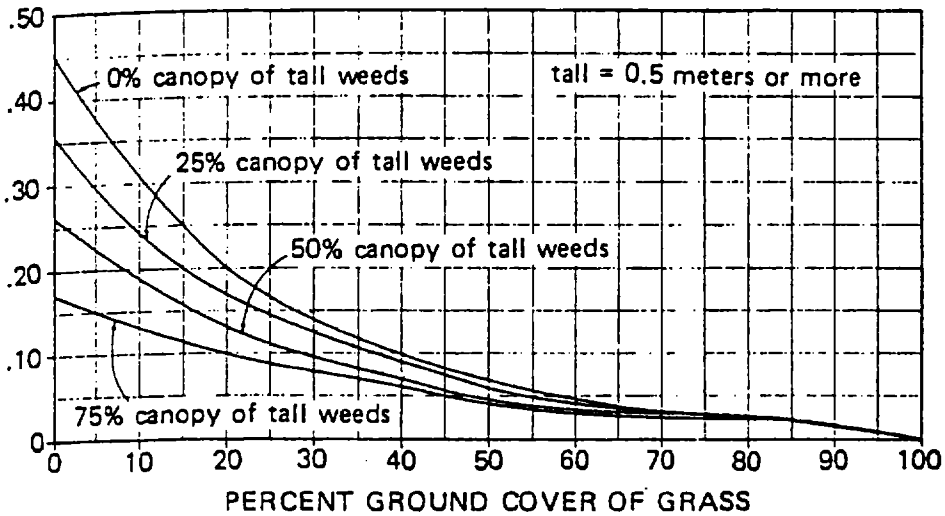
For the 5% to 4H:1V - Segment 2 of the Combined Slope:

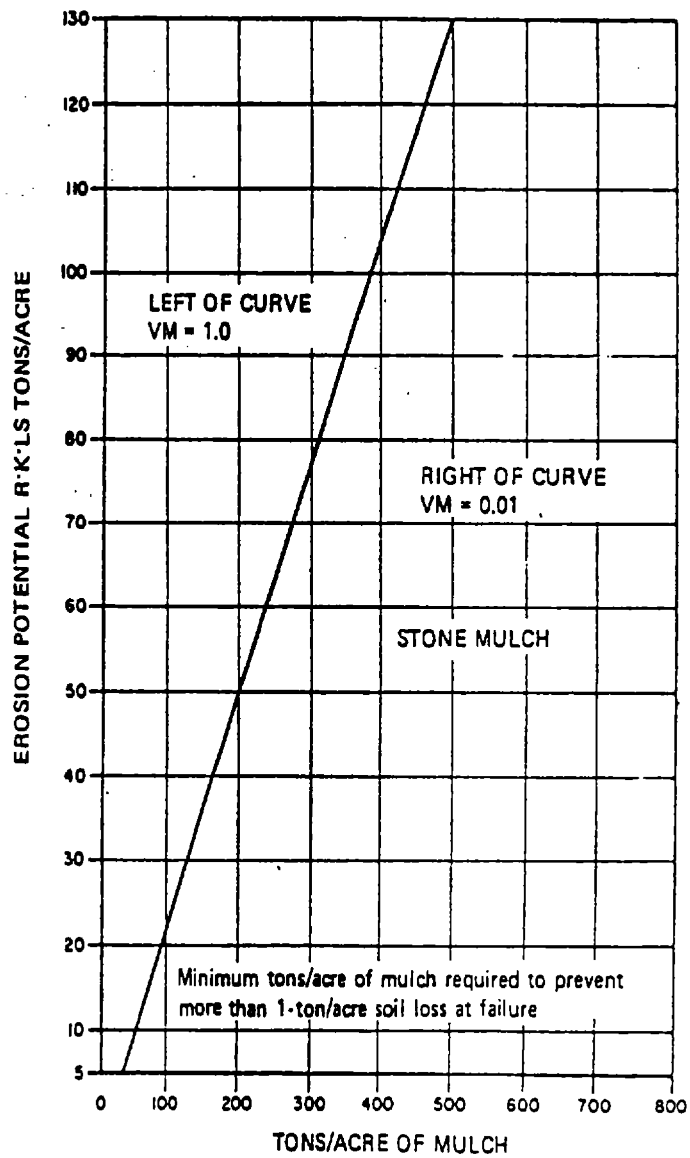
VM = 1/135.37 = 0.007

Percent Ground Cover of Grass = 95% (Regardless of tall weeds)

HANSEN ALLEN & LUCE INC ENGINEERS

CLIENT:	Wasatch Regional	SHEET	9 OF 9
PROJECT:	Landfill Permit	COMPUTED:	GLJ
FEATURE:	Erosion Protection	CHECKED:	KCS
PROJECT NO.:	113.30.100	DATE:	December 2004





STATE OF UTAH
DEPARTMENT OF NATURAL RESOURCES

Technical Publication No. 42

HYDROLOGIC RECONNAISSANCE OF THE NORTHERN GREAT SALT LAKE DESERT AND SUMMARY HYDROLOGIC RECONNAISSANCE OF NORTHWESTERN UTAH

by

Jerry C. Stephens, Hydrologist

U. S. Geological Survey

Prepared by

the United States Geological Survey

in cooperation with

the Utah Department of Natural Resources

Division of Water Rights

1974

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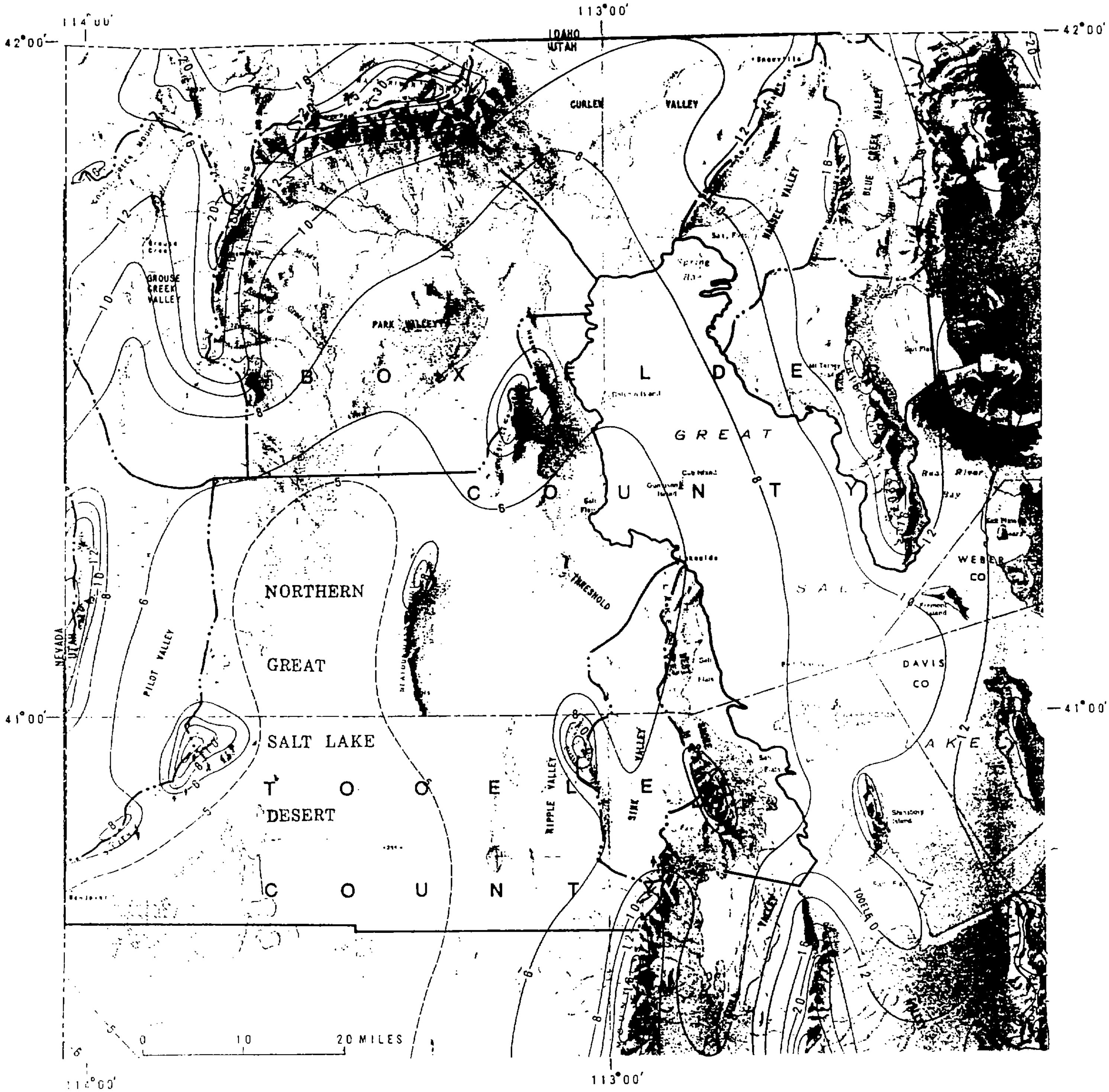

EXPLANATION

———5————

Line of equal normal annual precipitation, 1931-60; interval 2 and 4 inches; supplementary 5-inch line and adjusted lines near State line dashed

Drainage divide
Arbitrary boundary
} Hydrologic subarea boundary

Base from U.S. Geological Survey 1959
State of Utah 1:500,000 shaded relief

Climatologic data from U.S. Weather Bureau (1963). Adjusted along State line to reconcile with Nevada data (Hardman, 1965) by J. W. Hood and J. C. Stephens

Figure 1.—Map showing location, physiography, precipitation, and hydrologic subarea boundaries of northwestern Utah.

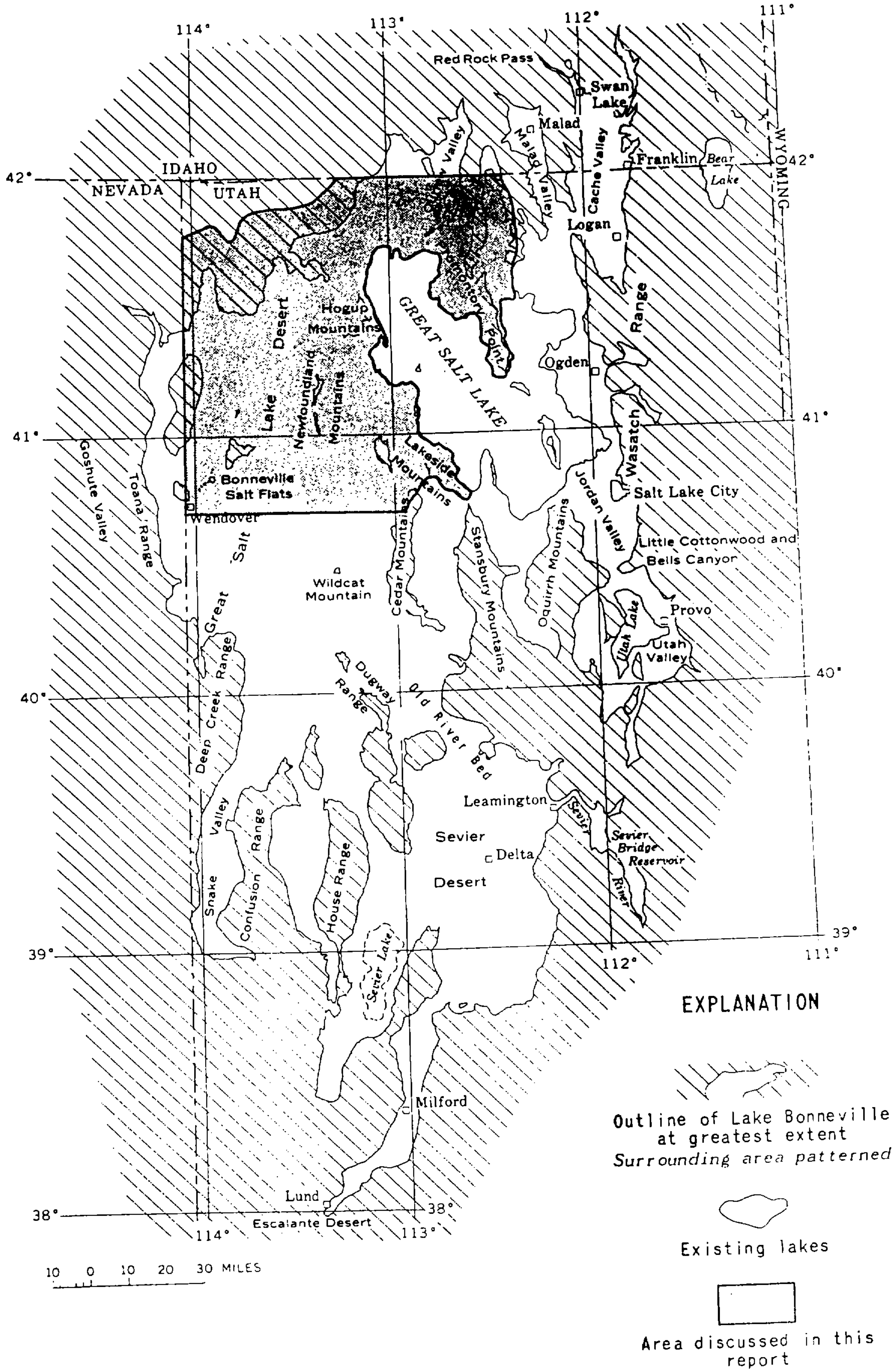

Figure 2.—Map of Lake Bonneville (after Crittenden, 1963).

Table 3.-Estimated average annual volumes of precipitation and ground-water recharge in the northern Great Salt Lake Desert
(Areas of precipitation zones measured from isohyetal and geologic maps, figure 1 and plate 1)

Precipitation zone (inches)	Locality	Area (acres)	Precipitation		Recharge	
			Feet	Acre-feet	Percent of precipitation	Acre-feet
		Consolidated rocks and alluvium				
8-more than 12	West slope Grassy Mountains	7,810	0.88	6,870	8	550
Do	East slope Silver Island Range	10,880	0.88	9,570	8	770
8-more than 10	Terrace and Hogup Mountains	19,260	0.80	15,410	8	1,230
6-more than 8	Newfoundland Mountains	9,020	0.63	5,680	3	170
6-8	Periphery of northern Great Salt Lake Desert	91,650	0.58	53,150	3	1,590
5-6	Flanks of Newfoundland Mountains	24,700	0.46	11,360	2	230
Subtotal		163,320		102,040		4,540
		Lakebed deposits and dune sand				
6-8	Periphery of northern Great Salt Lake Desert	14,530	0.58	8,430	0	0
5-6	Floor of northern Great Salt Lake Desert	648,000	0.46	298,000	0	0
Less than 5	Central part of northern Great Salt Lake Desert	431,000	0.40	172,000	0	0
Do	Bonneville Salt Flats (crystalline salt beds)	96,000	0.40	38,400	(1/)	20,000
Subtotal		1,189,530		516,830		20,000
Total (rounded)		1,350,000		620,000		25,000

1/ See page 13 for discussion of recharge estimate for crystalline salt beds.

HYDRATION OF GCLs ADJACENT TO SOIL LAYERS

An extensive laboratory testing program was undertaken to investigate the potential for hydration of a GCL when placed against a compacted soil layer. Three different GCLs were used to evaluate the effects of hydration time, initial GCL water content, thickness of soil layer and overburden pressure.

Tests were conducted using a low plasticity clay, commonly found in the Cincinnati, Ohio area. Specimens of GCL with a known moisture content, were placed in a specially designed test apparatus, where a soil with a known moisture content was compacted into the base and the GCL was placed on top. The specimen was then loaded with a load platen and allowed to hydrate for a specific amount of time. At the end of the hydration period, the GCL was tested for moisture content. The GCL was left in contact with the soil for periods of 5, 25 and 75 days to define the effect of test duration on the hydration of the GCL.

Test results show that significant increases in the moisture content of a GCL may occur in the first few days of a GCL's contact with a soil stratum. Overburden pressures within the range tested (i.e. 5 to 390 kPa) did not deter the hydration process, but a larger soil thickness resulted in a larger increase in GCL moisture content.

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EPA/600/R-96/149
June 1996

EPA

Report of 1995 Workshop on Geosynthetic Clay Liners

HYDRATION OF GCLs ADJACENT TO SOIL LAYERS

Overview of Testing Program

The authors conducted an extensive laboratory testing program to evaluate the potential for hydration of GCLs placed against a compacted subgrade soil layer. Hydration tests were performed on three different GCL products to evaluate the effects of: (i) test duration (i.e., hydration time); (ii) soil initial water content; (iii) thickness of soil layer; and (iv) overburden pressure. Three commercially-available GCL products, namely, Claymax®, Bentomat®, and Bentofix® were used in the testing program. The soil used in the testing program was obtained from the USEPA GCL Field Test Site at the ELDA-RDF facility in Cincinnati, Ohio. This material is classified as low plasticity clay (CL) based on the Unified Soil Classification System (USCS). Tests were performed on two different soil samples and consistent results were obtained between samples. The results reported herein were obtained from tests on a sample with 99 percent of the soil passing the U.S. No. 200 standard sieve and 33 percent smaller than 2 μm (clay fraction). The liquid limit of the soil is 41 and the plasticity index is 19. The soil has an optimum moisture content (OMC) of 20 percent and a maximum dry unit weight of 16.7 kN/m^3 based on the standard Proctor compaction method (ASTM D 698).

Testing Apparatus and Procedure

Figure 11 shows the apparatus specially designed to conduct the GCL hydration tests. The apparatus consists of a polypropylene mold 75 mm in diameter and 150 mm in height. A geomembrane/GCL/soil composite specimen is placed in the mold and covered with two layers of a thin vapor barrier. A loading platen is placed on the specimen for application of overburden pressure.

To process the soil, it was first passed through a U.S. No. 4 standard sieve. The soil was then moisture conditioned to achieve the desired moisture content. The moist soil was placed in the mold in a loose condition and statically compressed to 50-mm thick lifts. The soil was compacted to a dry unit weight equal to approximately 90 percent of the maximum dry unit weight based on the standard Proctor method (ASTM D 698). Two soil lifts were used giving a total thickness of 100 mm. The GCL and geomembrane specimens were carefully trimmed from the same sheets. The initial moisture content of the GCL was measured by taking a small sample from the same GCL sheet and measuring its weight before and after oven drying. The initial moisture content of the GCLs varied between 15 and 20 percent.

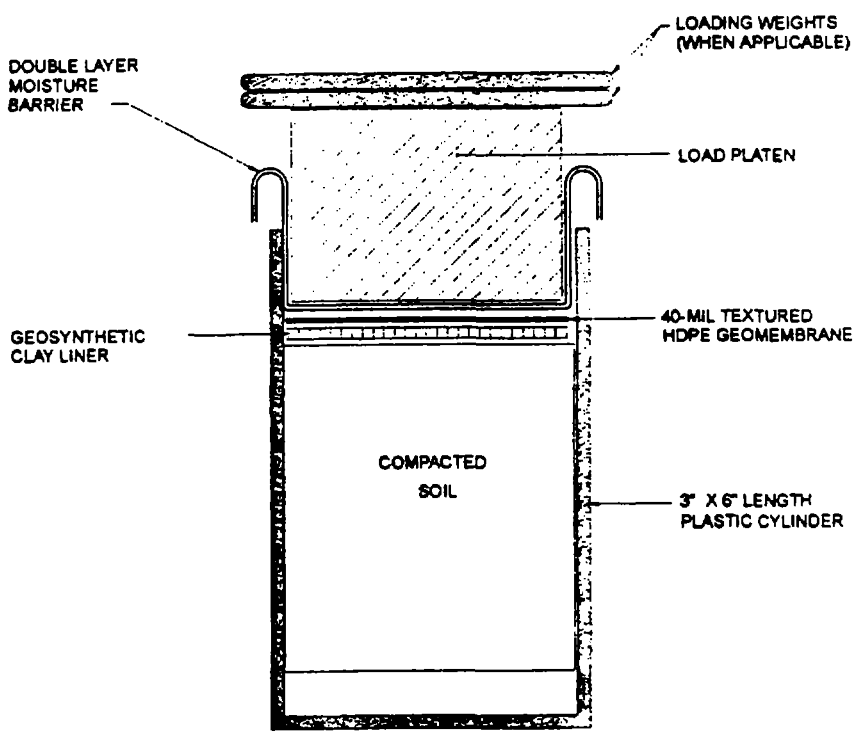

Figure 11. Simplified diagram of GCL hydration test set-up.

The GCL and geomembrane were placed on the soil and covered with the vapor barrier. The side of the GCL placed against the soil was woven in the case of Claymax® and nonwoven for Bentomat® and Bentofix®. Overburden pressure of 10 kPa was applied on the composite specimen utilizing standard weights which were placed on the loading platen. The entire apparatus was then placed in a temperature and humidity controlled room for the desired hydration time period. At the end of the hydration period, the test specimen was removed and the water content of the GCL and soil were measured. The final moisture content of the GCL was measured by weighing the entire GCL specimen before and after oven drying. The final moisture content of the soil was measured as the average water content of three samples obtained from the top, middle, and bottom of the soil specimen.

Testing Conditions and Results

As previously described, test conditions were varied to evaluate the effects of several factors on the hydration of GCLs. To evaluate the effect of test duration, tests were performed where the GCL was in contact with the soil for 5, 25, and 75 days. Soil specimens were compacted to initial moisture contents equal to OMC, 4 percentage points dry of OMC, and 4 percentage points wet of OMC to evaluate the effect of soil initial moisture content on GCL hydration.

Figures 12, 13, and 14 present the results of the hydration tests for the GCL products Claymax®, Bentomat®, and Bentofix®, respectively. These figures show that the moisture content of all three GCLs increased significantly as a result of contact with compacted subgrade soil. The increase in GCL water content was significant after only five days of hydration. With increasing time, GCL water content continued to increase at a decreasing rate. For most tests, GCL water content reached a maximum value after about 25 days of soil contact and for some of the tests water content continued to increase even after 75 days of hydration. It is interesting to note that all three GCL products showed relatively similar behavior. Increases in water content were comparable for the three GCL products despite differences in GCL fabric (i.e., woven vs. nonwoven) and types of bentonite clay used to manufacture the GCLs.

Figures 12, 13, and 14 illustrate the influence of soil subgrade initial moisture content on the hydration of GCLs. From these figures, it is evident that the moisture content of the GCL for any particular hydration time increases as the initial moisture content of the soil increases. These figures also show that a small increase in soil initial moisture content can have a significant impact on GCL moisture content. For example, after 75 days of hydration, the moisture content of Claymax® was approximately 16 percent higher when the initial moisture content of the soil was equal to OMC than when it was 4 percentage points drier than OMC. This behavior is expected because more water is available in the soil for the GCL to hydrate.

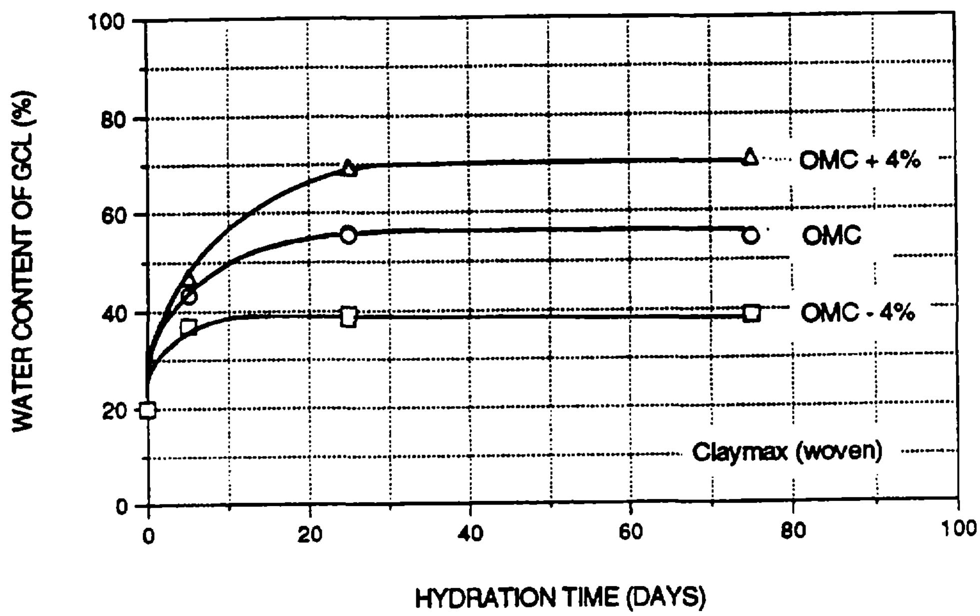

Figure 12. Increase in GCL moisture content due to contact with compacted subgrade soil: Claymax® with woven geotextile against soil.

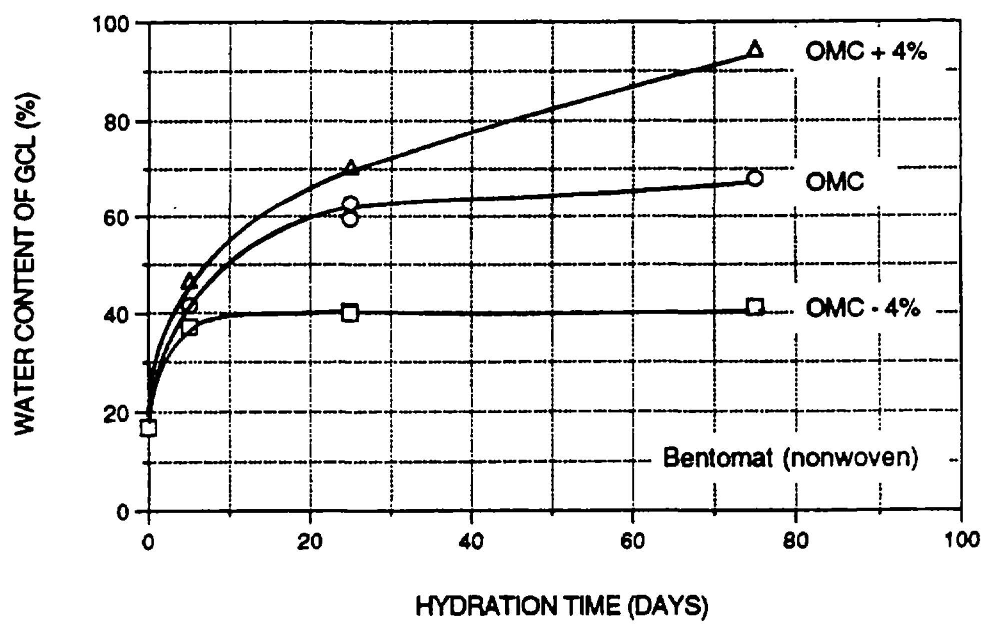

Figure 13. Increase in GCL moisture content due to contact with compacted subgrade soil: Bentomat® with nonwoven geotextile against soil.

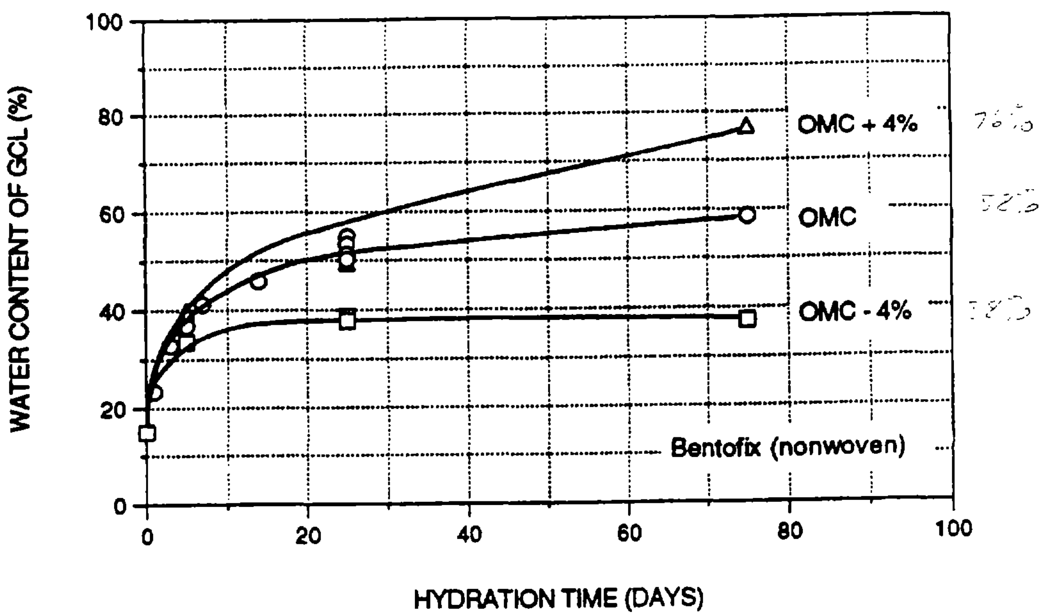

Figure 14. Increase in GCL moisture content due to contact with compacted subgrade soil: Bentofix® with nonwoven geotextile against soil.

The examination of the curves shown in Figures 12, 13, and 14 shows that the time required for the GCL to reach its final moisture content is less in the case of a dry soil than in the case of a wet soil. At the lowest soil initial moisture content tested, GCL moisture content ceased to increase after about 5 to 25 days. At the highest initial moisture content tested, the Bentomat® and Bentofix® GCLs continued to increase in moisture content after 75 days of hydration.

To evaluate the effect of soil layer thickness, specimens were prepared using 50, 100, 150, and 200 mm of soil thickness. Soil initial moisture content was 20 percent and dry unit weight was 14.9 kN/m³ for all specimens. Figure 15 shows the results of hydration tests for the Bentofix® GCL after 25 days of hydration. The GCL moisture content increased with the increase of the soil layer thickness. However, it appears that only a small change in moisture content increase occurs for thicknesses greater than 100 mm.

The effect of overburden pressure on GCL hydration is illustrated in Figure 16 for the Bentofix® GCL. As shown in this figure, overburden pressure in the range of 5 to 390 kPa did not significantly affect the rate of GCL hydration during the 25-day test duration.

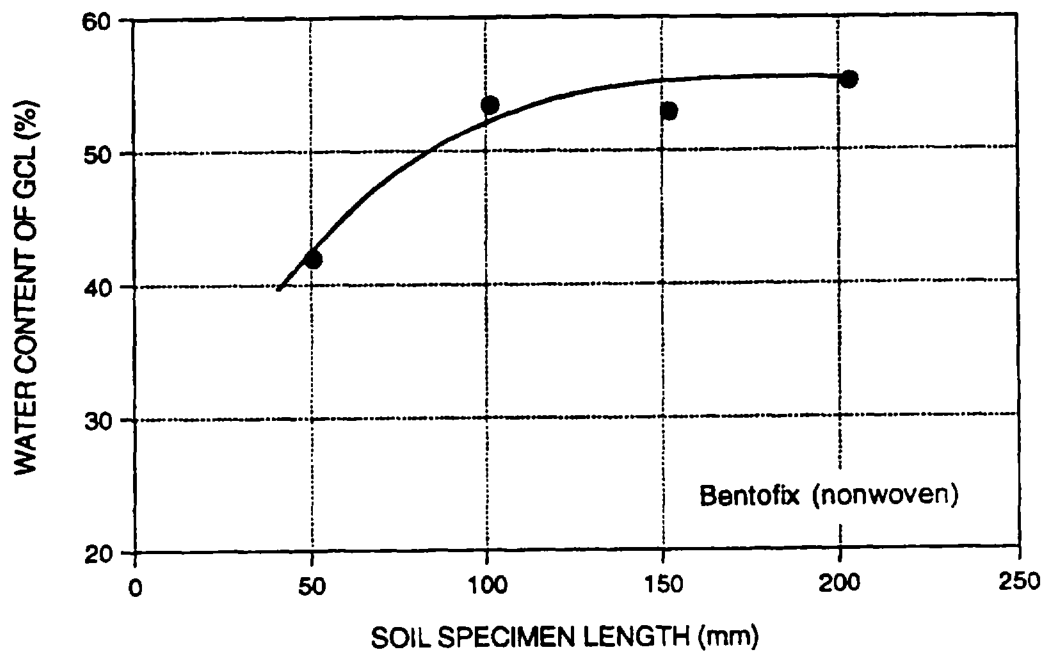

Figure 15. Influence of subgrade soil layer thickness on GCL moisture content.

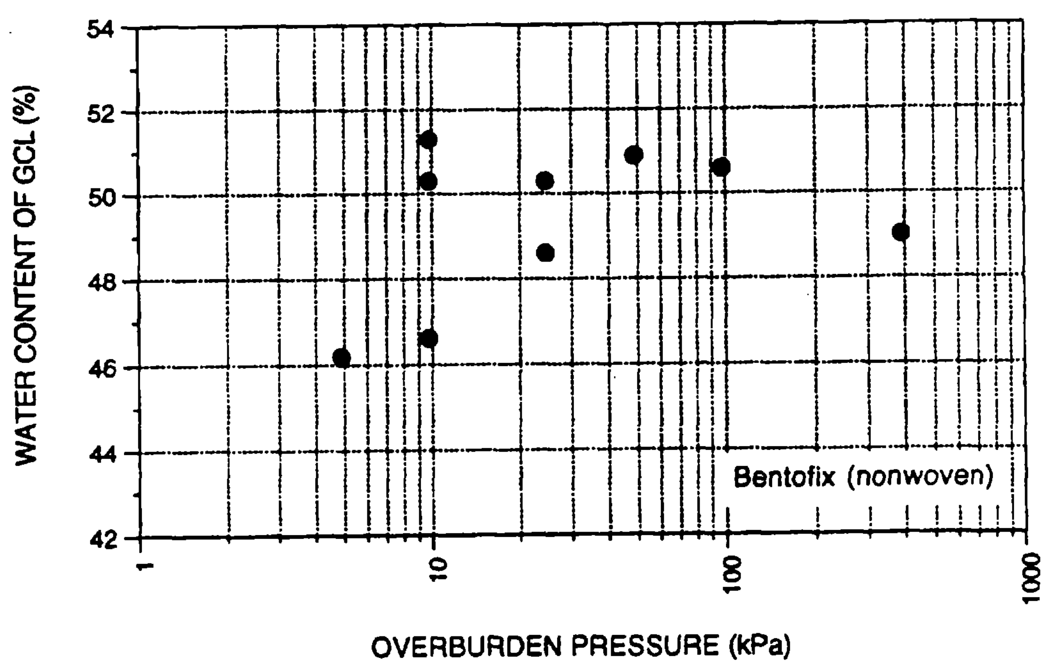

Figure 16. Influence of overburden pressure on the increase in GCL moisture content.

Summary

From the testing program results described above, the following can be concluded:

- GCLs will hydrate when placed in contact with subgrade soils compacted within the range of moisture contents typically found in earthwork construction specifications; this conclusion is consistent with data provided by Daniel et al. [1993]; even for the driest soil (compacted 4 percentage points dry of OMC), GCL moisture contents consistently increased from an initial value in the range of 15 to 20 percent up to about 40 percent within a 100-day period; it should thus be anticipated that GCLs placed even against relatively dry compacted subgrades will undergo substantial hydration;

- given that Daniel et al. [1993] have shown that long-term GCL shear strengths are insensitive to water content for water contents above about 50 percent, stability analyses involving GCLs placed in contact with compacted subgrade soils should be based on hydrated GCL shear strengths;

- significant increases in GCL moisture contents may occur within a few days of GCL contact with a moist soil; the rate of GCL hydration is initially highest and then decreases with increasing time;

- within the range of conditions tested a higher soil moisture content results in a higher GCL moisture content;

- larger soil layer thickness results in a larger increase in GCL moisture content, however, for soil layer thicknesses greater than 100 mm only insignificant increases were observed with increasing soil layer thickness;

- overburden pressure within the range tested (i.e., 5 to 390 kPa) did not influence the hydration process; and

- differences between GCL products tested (i.e., type of bentonite clay and fabric) did not seem to significantly affect the test results.

TECHNICAL SERVICES REPORT
(LINING TECHNOLOGIES)

Lab Report No:	19/96	
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Analysed By:	J. Burrows	
Date Reported:	24/10/96	
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Use of the Oedometer to Determine the Confinement Provided by Bentomat Needlepunch Reinforcement

(1) Introduction. The oedometer is normally used to determine one-dimensional consolidation (vertical settlement). Generally the one-dimensional consolidation test is used for the determination of the consolidation characteristics of soils of low permeability. Tests are usually carried out on specimens prepared from undisturbed samples. Data obtained from these tests, together with classification data and a knowledge of the soils history, enables estimates to be made on the behaviour of foundations under load.

For the purposes of this work, however, the oedometer was used in reverse, to investigate the swell of bentonite granules under various confining forces. Water was introduced to specimens already under load and the swell (vertical displacement) was measured with time.

Additionally, a sample of Bentomat was tested, with no external load applied. Therefore, an estimate of the confinement due to the needlepunching alone could be made after comparison with the swell of the bentonite granules alone.

(2) Summary. The swell of bentonite granules over a given time is reduced as the confining force is increased. Comparison of the Bentomat test results, with those obtained for bentonite granules alone, indicated a confinement due to the needle-punching of 10.7 KPa (equivalent to approximately 500 mm of cover material).

(3) Experimental. An mass of bentonite granules (equal to that in Bentomat) in "as received" conditions were lightly "tamped" into the cutting ring. Confining forces of O KPa, 10 KPa and 20 KPa were used. The loads were applied first, then the vessel was filled with deionised water. Displacement (swell) was monitored with time for each load.

For comparative purposes, a sample of Bentomat was also cut to fit the cutting ring and then tested with a zero load applied in the same way. The results are shown in Figures 1 and 2. Interestingly, displacement reaches a plateau after approximately one week.

(4) Conclusion. The oedometer can be used to estimate the confining force due to needle-punching in Bentomat. The sample of bentonite under a load of 10kPa exhibited the same amount of swell and same hydrated thickness as the sample of Bentomat under no load. Thus it can be concluded that the needlepunch reinforcement of Bentomat provides approximately 10kPa of confining stress within the bentonite layer. This is equivalent to approximately 500 mm of cover material.

K. Harris.

FIGURE 1 : THE SWELL OF GRANULAR BENTONITE UNDER VARIOUS CONFINING FORCES.

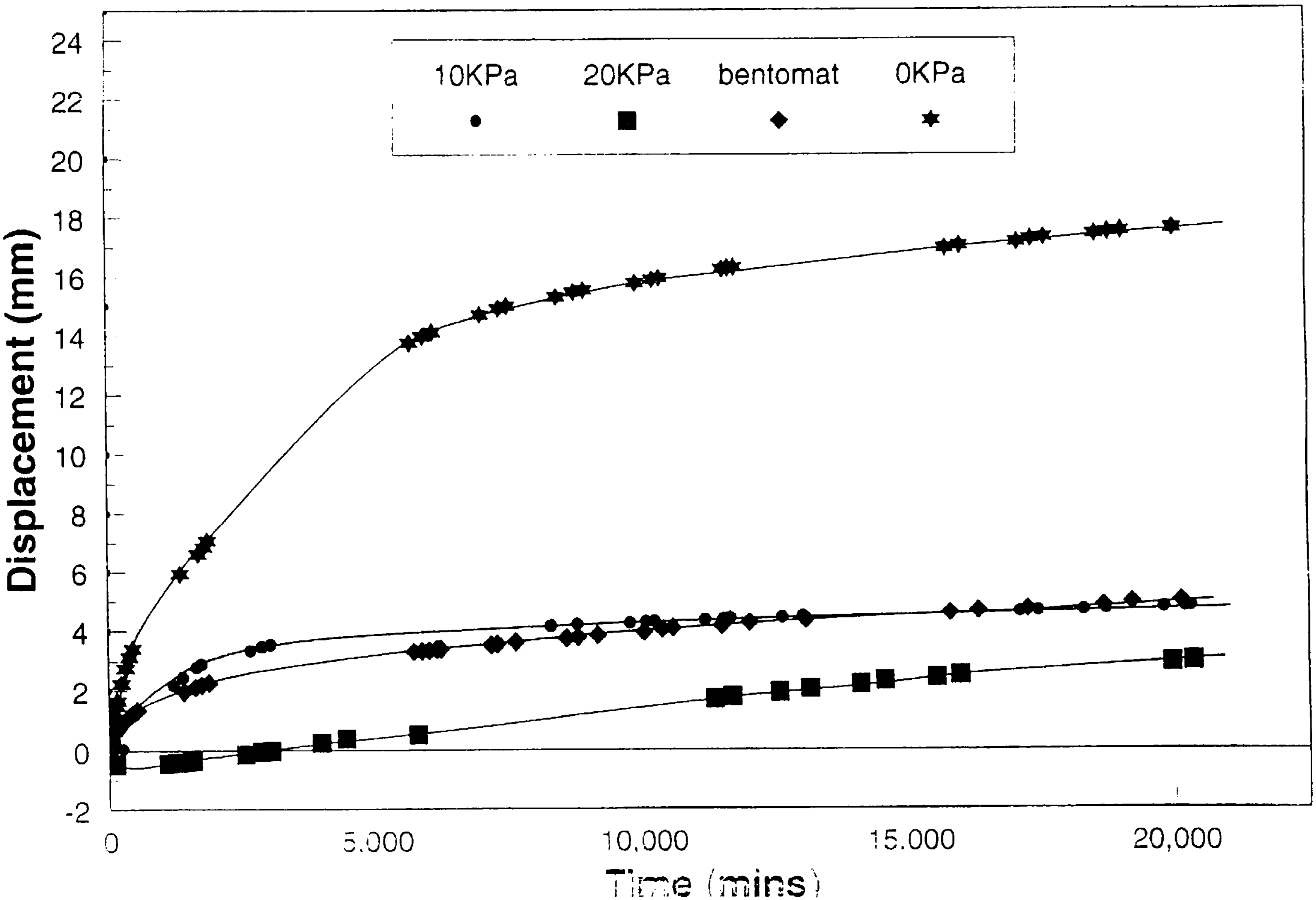

FIGURE 2 : THE EFFECT OF CONFINING FORCE ON THE SWELL OF BENTONITE GRANULES